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LOW TEMPERATURE OXIDATION DITCH FIELD STUDY

by

NORBERT WALTER SCHMIDTKE

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

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MASTER OF SCIENCE

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## UNIVERSITY OF ALBERTA

## FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled LOW TEMPERATURE OXIDATION DITCH FIELD STUDY submitted by NORBERT WALTER SCHMIDTKE in partial fulfilment of the requirements for the degree of MASTER OF SCIENCE.





## ABSTRACT

Results of a field investigation into the behavioral characteristics of an oxidation ditch treating domestic wastes from the Town of Olds, Alberta, Canada, are presented. The object of this study was to assess the winter operation of this system during ambient temperatures as low as  $-22^{\circ}$  Fahrenheit.

During the study period which extended from October 10th, 1966 to February 20th, 1967 routine analyses of waste influent, treatment plant effluent and ditch mixed liquor were performed.

The study showed that BOD-5 removal efficiencies of 85 percent were achieved. Waste influent COD concentrations of 542 mg/l were reduced to an average value of 218 mg/l in the plant effluent. The residual total suspended solids in the effluent decreased from 96 mg/l to 70 mg/l as the mixed liquor suspended solids in the ditch increased from 2260 mg/l to 3400 mg/l. Phosphate removals as measured by the orthophosphate test average 20 percent. Nitrification at liquid temperatures of  $+2^{\circ}$  Centigrade was found to be negligible for the duration of the testing program.

Special studies consisting of effluent coliform concentration determinations, mixed liquor and effluent oxidation-reduction potential, mixed liquor settleability, ditch liquid velocities, mixed liquor and effluent dissolved oxygen concentrations, as well as oxygen transfer rate coefficient and oxygen utilization by the waste were performed.



Effluent coliform concentrations averaged 2000 organisms per ml. The ditch liquid velocity studies show that at a volumetric rotor loading of 12,800 l.gal/ft. of rotor the average daily mean velocity is 1.54 ft/sec.

Oxygen transfer capacities of the rotor were measured and compared to the rotor manufacturer's ratings. It was found that the oxygenation capacity of the rotor was approximately 50 percent of the manufacturer's rating. Actual oxygen requirements for the waste were determined as 1.43 lbs. oxygen per lb. BOD-5 removed.

McKinney's mathematical analysis for complete mixing activated sludge was used to compare measured and calculated BOD-5 effluent values. It was found that at liquid temperatures of +2° Centigrade the theoretical BOD-5 values were approximately 20 mg/l and actual values 40 mg/l. Calculated volatile suspended solids for the effluent were 125 mg/l while the measured effluent volatile suspended solids were 55 mg/l. The disagreement between measured and theoretically derived effluent values suggests further research of k-factors for complete mixing activated sludge systems under liquid temperatures of less than +5° Centigrade is required. For low temperature conditions a correlation factor of 2 appears to be applicable between theoretical and actual BOD-5 effluent concentrations.

These studies show that the oxidation ditch is not noticeably influenced by low temperatures and is a suitable method for waste treatment in a low temperature environment.





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## GLOSSARY OF TERMS

- ASSIMILATION    The absorption and utilization of a nutrient by an organism.
- BASAL METABOLISM    The quantity of energy required to support the life processes of any organism at rest and in a non-growing state.
- BIOCHEMICAL OXYGEN DEMAND    The amount of oxygen required for the biological oxidation of the organic matter in a liquid.
- BIOLOGICAL OXIDATION    A biochemical reaction in which materials combine with oxygen to produce energy.
- CATABOLISM    Those chemical and physical processes continuously taking place in living organisms and cells by which protoplasm is used and broken down into simpler substances with the release of energy.
- CHEMICAL OXIDATION DEMAND    The amount of oxygen required for the chemical oxidation of organics in a liquid.
- COLIFORM ORGANISMS    A group of bacteria recognized as indicators of fecal pollution.
- DENITRIFICATION    A biological process in which gaseous nitrogen is produced from nitrite and nitrate.
- ECOSYSTEM    Any segment of nature in which there is an exchange of materials between living organisms or dead organisms and their environment.
- ENDOGENOUS RESPIRATION    An auto-oxidation of cellular material that takes place in the absence of assimilable organic material to furnish the energy required for the replacement of worn out components of protoplasm.



HYDROLYSIS	A chemical reaction in which a compound reacts with the ions of water ( $H^+$ and $OH^-$ ) to form a weak acid, a weak base, or both.
METABOLISM	The chemical and physical processes continuously taking place in living organisms and cells, comprising those processes by which assimilated nutrient is built up into protoplasm (anabolism), and those by which protoplasm is used and broken down into simpler substances, with the release of energy (catabolism).
MICRO-ORGANISMS	Microscopic plants and animals such as bacteria, molds, protozoa, algae and small metazoa.
MIXED LIQUOR	Mixture of activated sludge and liquid waste.
NITRIFICATION	A biological process in which ammonia is converted first to nitrite and then to nitrate.
NITROGEN FIXATION	The conversion of gaseous elementary nitrogen into forms of nitrogen available to organisms.
NUTRIENT	Any substance assimilated by organisms which promotes growth and replacement of cellular constituents.
PHOTOSYNTHESIS	The mechanism by which chlorophyll-bearing plants utilize light energy to produce carbohydrate and oxygen from carbon dioxide and water.
PROTOPLASM	A semi-fluid colloidal material comprising the bulk of all animal and plant cells and consisting largely of water, proteins, fats, carbohydrates, and inorganic salts.
PSYCROPHYLIC	The ability of an organism to grow at 0 degrees Centigrade.
RESPIRATION	The mechanism by which living organisms utilize organic materials for the production of energy.
SAPROPHYTIC	The ability of an organism to live on dead organic matter.
VOLATILE SOLIDS	Organic fraction of solids.





## CHAPTER I

### INTRODUCTION

Man is his own worst enemy. This is readily manifested when examining the effects of indiscriminate discharge of waste products to air, land and water which results in the pollution and ultimate destruction of his environment.

Nature has an inherent ability, specific capacity and a fixed rate of waste product assimilation. In most cases this is achieved by way of biological oxidation of organic materials which are composed of relatively few elements. How each element is normally transformed in nature can best be illustrated by way of FIGURE 1 which outlines the route followed by carbon in a typical ecosystem. Similar paths of ecosystems exist for other elements.

Since nature's process of degrading organic materials can neither be hurried nor overtaxed (thereby becoming incapable of keeping pace with the rate of pollutant discharge), sanitary engineers have created controlled biological environments which are amenable to the bio-degradation of organic materials. These environments are known as pollution control centres. In essence, an attempt is made to duplicate nature, but the advantage lies in that by providing optimum conditions for biologic growth, the rate of bio-degradation can be controlled and greatly accelerated.



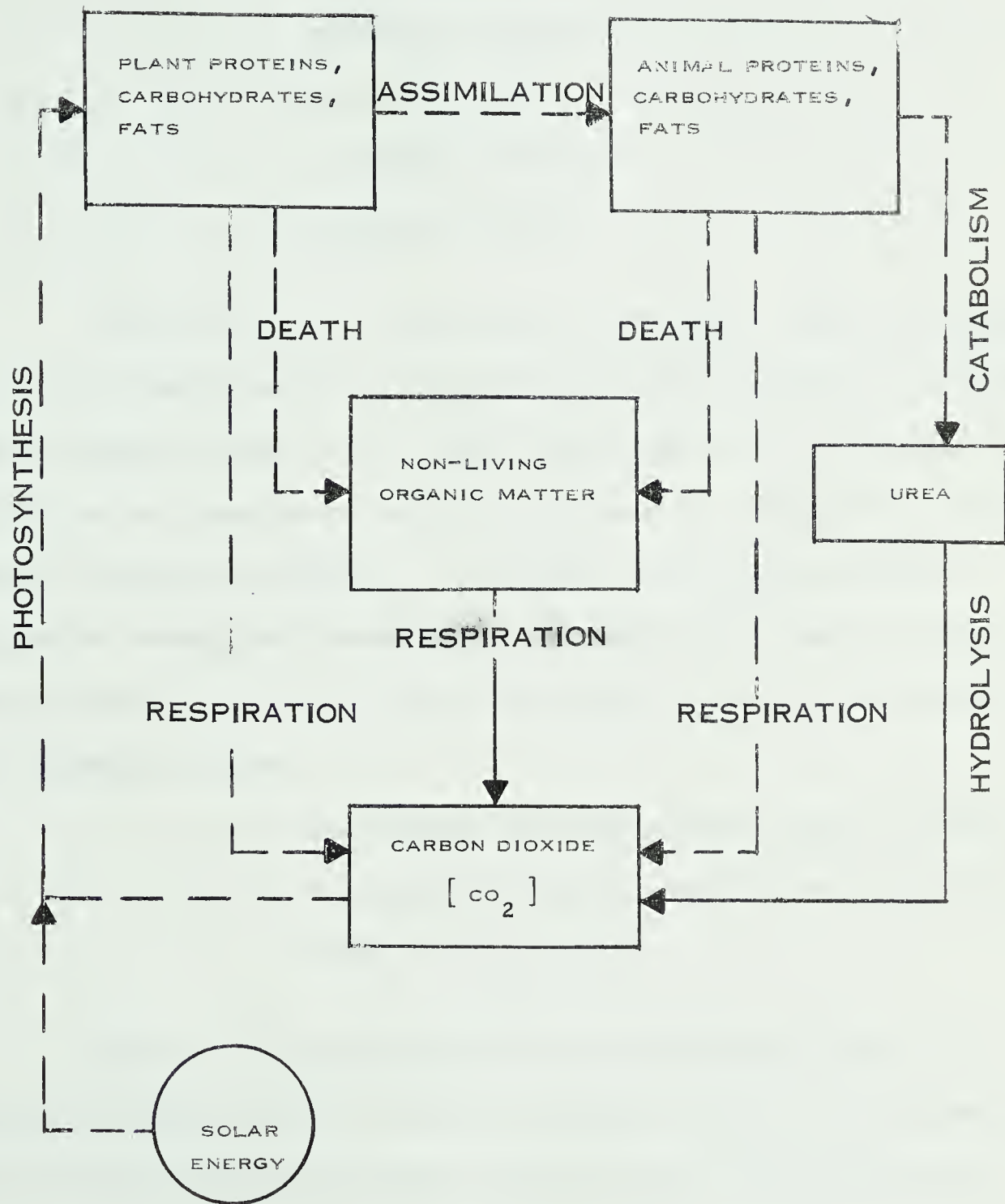


FIGURE : I CARBON CYCLE IN NATURE [ AFTER RICH, 1963 ]



Five basic biological treatment systems of varying efficiency are available as pollution control tools:

1. aerobic digestion
2. anaerobic digestion
3. oxidation ponds
4. trickling filters
5. activated sludge

The basis for all five systems is the same. Micro-organisms utilize the organic matter in the wastes for the production of new cells and for energy to remain alive. The organic matter so converted into new cells is not completely stabilized but merely transformed from one chemical structure to another. The organic matter which has been utilized for energy and converted into carbon dioxide and water has been completely stabilized. Hence, two problems exist in biological waste treatment systems:

1. the removal of organic matter from the waste water
2. the removal of micro-organisms which are present from the first phase

Because of its exceedingly high purification capacity, the activated sludge process is the most prevalent of the five aforementioned basic biological treatment systems used in areas of high population density, for large volumes or high concentrations of industrial wastes amenable to biological oxidation, or in areas where receiving streams have limited purification capacity.





The activated sludge process consists of contacting liquid wastes with a preformed flocculent assemblage of micro-organisms, non-living organic matter and inorganic materials in an aerated system. The end result is that a portion of the organics amenable to biological degradation is converted to additional activated sludge. The term "activated" stems from a unique property of the activated sludge. The floc surfaces are highly active in adsorbing colloidal and suspended materials from solution.

Because of high capital cost, activated sludge systems are normally limited to larger communities. In light of the emphasis on environmental pollution control the development of a high purification capacity - low capital cost waste treatment process for smaller communities would be most desirable. With this in mind the activated sludge process has been continually modified by changing such parameters as aeration period, suspended solids concentration, food to organism ratio, mode of aeration and distribution and rate of sludge return. To date (1967) approximately ten process variations are in existence, (Stewart, 1964)

One of the more recent (1953) modifications consists of a process known by a variety of names, such as Aeration, Oxidation or Pasveer Ditch Process. This process consists of complete mixing and extended aeration, meaning that the food to organism ratio is constant throughout the aeration vessel and the period of aeration is anywhere from one to three days.



The research presented herein evaluates the effect of extended residence time on oxidation ditch process efficiency under severe Canadian winter conditions.

More specifically, studies were conducted to ascertain the settleability of the activated sludge, oxidation ditch dissolved oxygen profiles, oxygen transfer rate coefficient and an oxidation-reduction potential (ORP) survey.



## CHAPTER II

### LITERATURE REVIEW

Noting that when sewage and air are in contact for prolonged periods putrescibility disappears, Dr. Smith of England devoted considerable attention to the aeration of sewage as early as 1882. But it was not until 1905 when Adeney performed aeration experiments that it became evident that by renewing the water-air interface, reaeration of oxygen depleted water would progress rapidly. In essence this finding paved the way for what is known today as the activated sludge process.

Various researchers experimented with different methods of reaeration and in 1917 the first activated sludge plant was built in Manchester, England. This plant used diffused aeration as the aerator mechanism. Experimentation with other aeration devices led to the use of paddles in an activated sludge plant in Sheffield, England. The function of the paddles as shown in FIGURE 2 is two-fold; to entrain air into the sewage and provide circulation to maintain the mixed liquor suspended solids from settling to the bottom of the aeration channel. The principle behind this aeration system was in line with Adeney's earlier experiments and conclusions that a rapid renewal of the water-air interface will enhance the rate of air entrainment and subsequent waste purification.





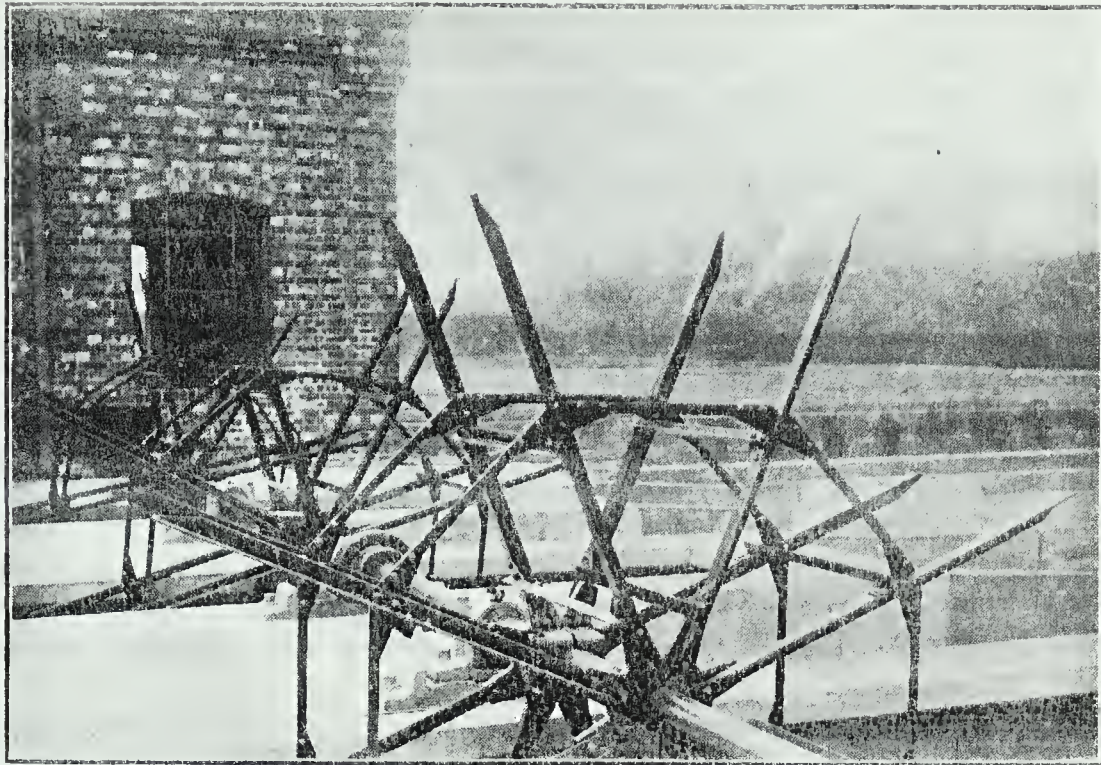


FIGURE : 2 EARLY PADDLE WHEELS AT SHEFFIELD, ENGLAND [ 1917 ]

Initially the paddles used had crossmembers thereby being similar in appearance to steamboat propulsion mechanisms. However, due to excessive wave action created within the aeration channels and subsequent splashing, the cross members were removed. The paddles were 10'-0" in diameter and 2'-6" in width rotating at a rate of 15 revolutions per minute. This rotational speed provided a horizontal velocity component of 1.7 feet per second (ft./sec.) which was found to be adequate in preventing the mixed liquor suspended solids (MLSS) from settling. Whatever mode of aeration tank agitation was employed it was noted that velocities from 0.8 ft./sec. to 2.0 ft./sec. were





required to keep the MLSS in suspension.

Based on experiences gained at Sheffield other similar activated sludge plants were constructed. In 1920 Milwaukee and Houston were among the first activated sludge plants constructed in the United States using a diffused aeration system. Meanwhile, in Toronto, Canada, Nasmith of the Toronto Health Department was carrying out waste purification experiments using paddles in small tanks in series. The function of his paddles was to entrain air into the sewage only and not to propel it. These examples are the fore-runners of the aeration devices which were developed by Kessener and became known as Kessener brushes (FIGURE 3).

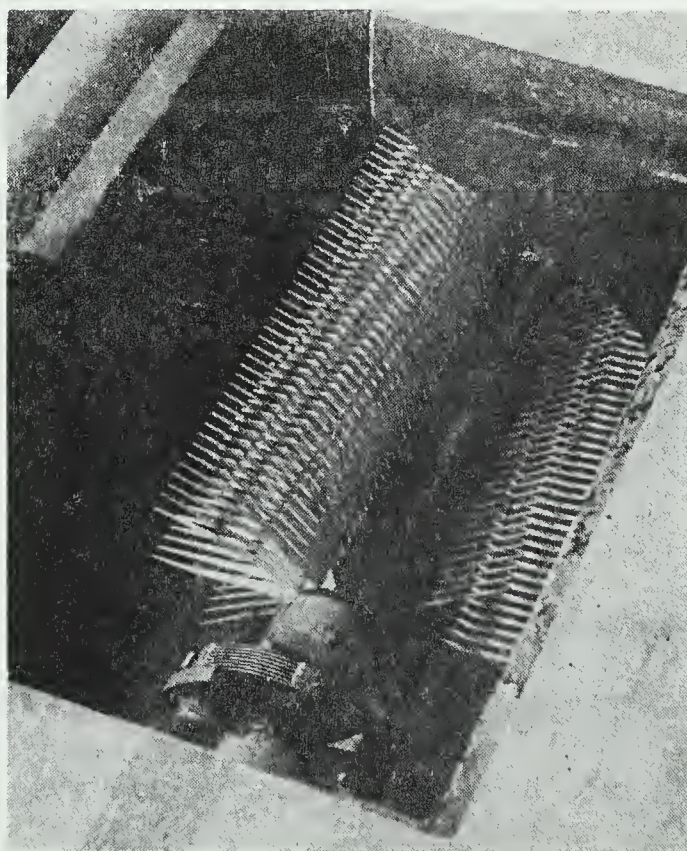


FIGURE : 3 KESSENER BRUSH



By 1932 Kessener and Ribbius (1934) developed a concept known as oxygenation-capacity (OC). This was to replace the aeration-capacity concept evolved and first presented mathematically by Adeney, making a comparison between different aeration systems possible. Adeney's concept was based on a calculated hypothetical aeration area whereas Kessener and Ribbius relate oxygenation-capacity to fixed reference conditions such as 10 degrees Centigrade, 760 mm barometric pressure, distilled water at zero dissolved oxygen (DO) content.

The results of experiments performed by Kessener and Ribbius show that for deaerated water a compressed air installation (porous plates, 3.5 m below liquid surface) would give an oxygen supply rate about four times greater than a surface aeration system. As indicated in FIGURE 4 the oxygen absorption curve for the compressed air installation exceeds the oxygen saturation value of the water at atmospheric pressure. This apparently is due to the air bubbles which are under pressure. Curves in FIGURE 4 showing the change of oxygen content in sewage for the compressed air and the surface aeration systems show that the maximum oxygen content for both systems is approximately equal. This indicates that the sewage influences oxygen absorption much more detrimentally with compressed air than with surface aeration.

In more recent studies, Pasveer (1951) employed with minor modifications the same principle for reaeration as Kessener and Ribbius did some 20 years earlier, and confirmed their observations that surface



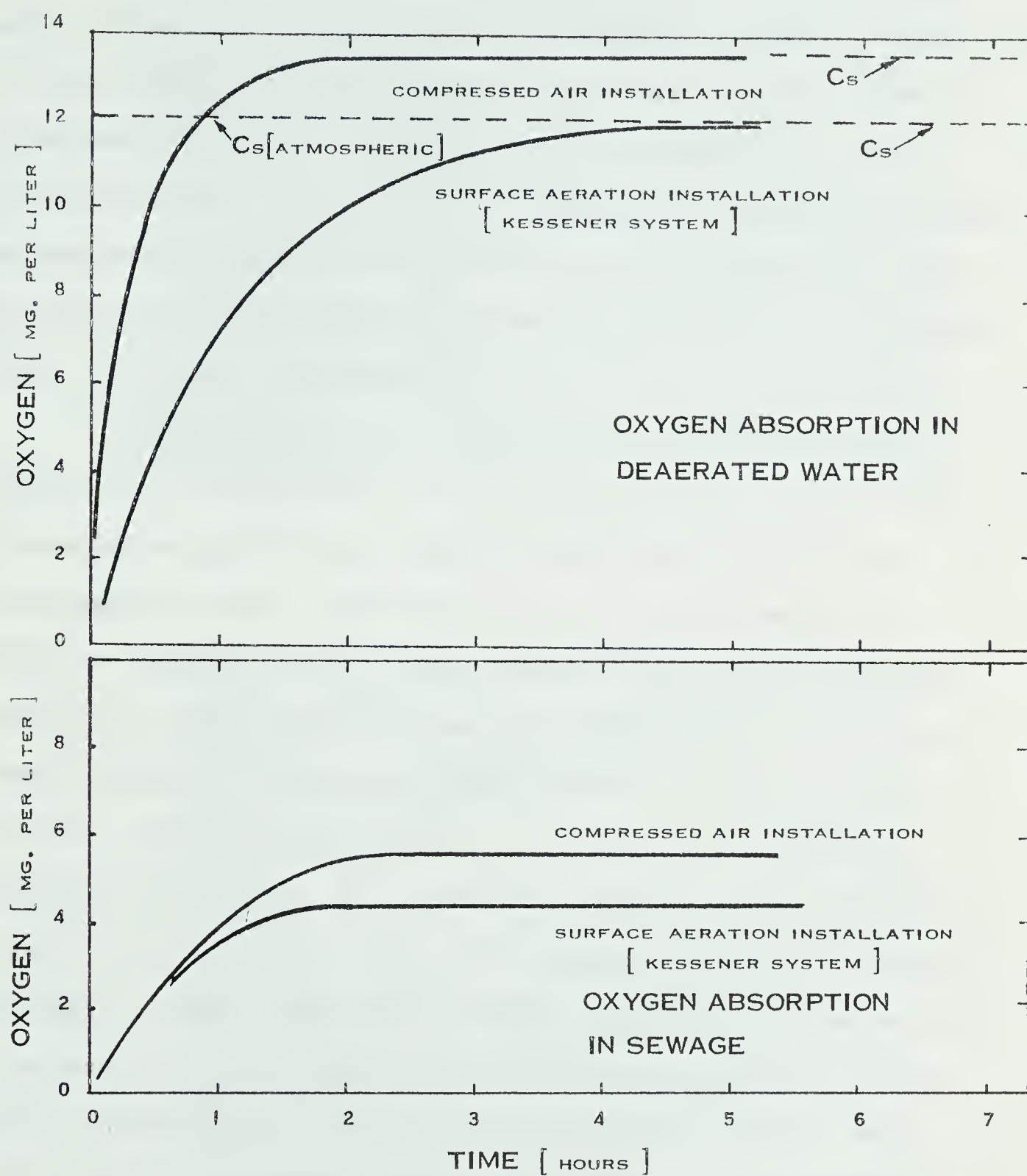


FIGURE : 4 OXYGEN ABSORPTION IN DIFFUSED AND SURFACE AERATION INSTALLATION.

[ AFTER KESSENER AND RIBBIUS, 1934 ]





creation and renewal of the water-air interface in the immediate vicinity of the brush are responsible for most of the oxygenation that occurs. This observation was made when comparing oxygenation capacities of large ( $936 \text{ ft}^3$ ) and small ( $141 \text{ ft}^3$ ) size tanks. Pasveer shows that in the smaller tanks the rate of oxygenation per unit of volume is increased in such a way that the oxygenation capacities per tank volume and per metre of brush remain the same or are even increased. From this follows that the aeration in a small brush-aerated tank proceeds just as well as in a large tank.

Laboratory studies of brush aeration by Baars and Muskat (1959) led to the development of two types of aeration rotors. One, a so-called angle-iron rotor (FIGURE 5) which may be used in the conventional activated sludge process and a so-called cage rotor (FIGURE 6) designed for use in the oxidation ditch. This development resulted in a two-to-four fold increase in OC over the conventional Kessener brushes. FIGURE 6 shows the geometric differences in construction between the angle-iron and cage rotor. Relationships as indicated on graphs a, b, and c point out the superiority of the cage rotor over the angle-iron rotor as an oxygenation device. For example, in FIGURE 7, graph c shows that for equal energy expenditures per gram of oxygen ( $\text{O}_2$ ) entrained (occurring at approximately 86 r.p.m., graph b) the angle-iron rotor would supply less than  $1600 \text{ g O}_2/\text{hr}$ . while the cage rotor would entrain approximately  $4400 \text{ g O}_2/\text{hr}$ . This represents a 175 percent aeration capacity increase of the cage rotor over the angle-iron rotor.





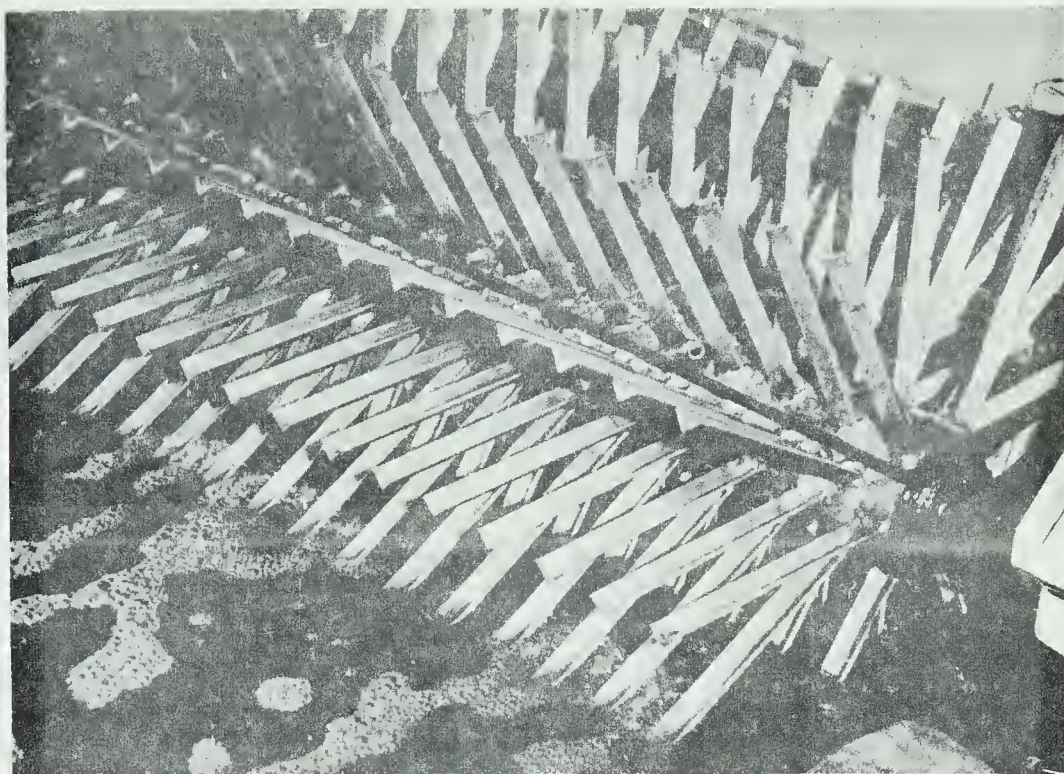


FIGURE : 5 ANGLE IRON AERATION ROTOR

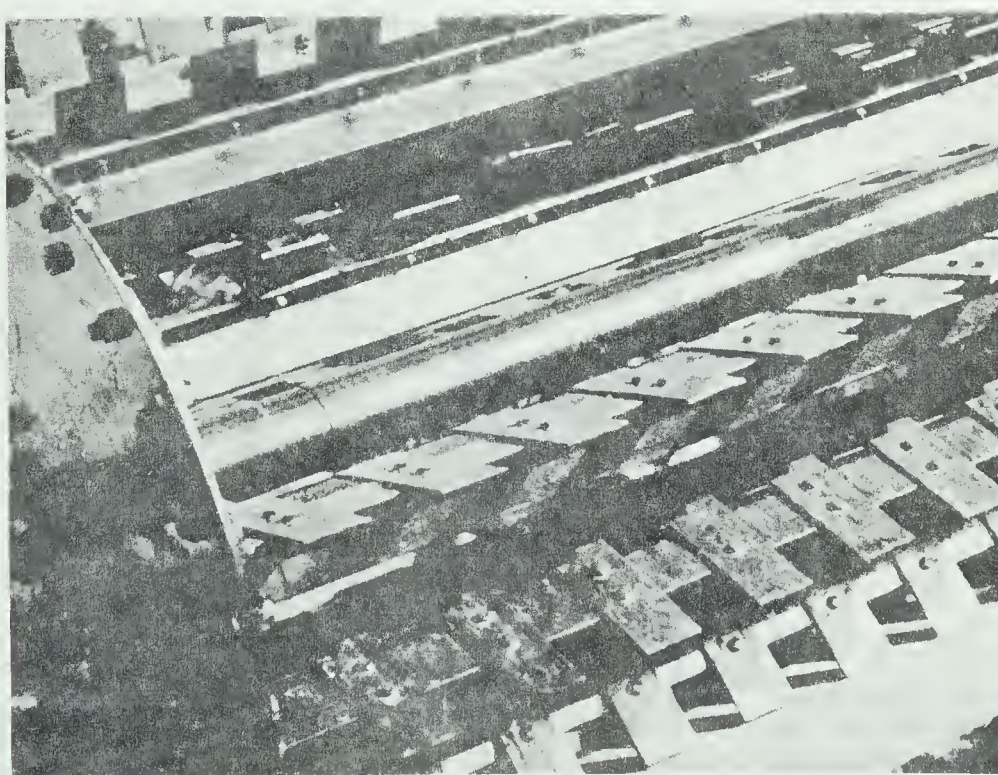
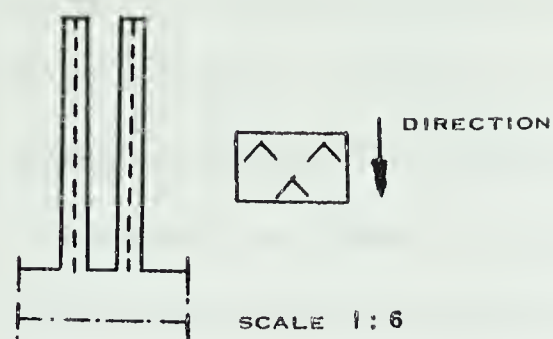


FIGURE : 6 CAGE AERATION ROTOR

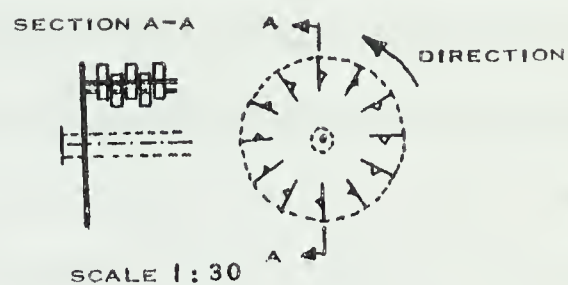




## ANGLE IRON AERATION ROTOR



## CAGE AERATION ROTOR



## TECHNICAL DATA

50 CM  
2 CM ANGLE STEEL  
1:1.5  
STAGGERED  
16 CM  
1

DIAMETER OF ROTOR  
SHAPE OF TOOTH  
TOOTH-TO-INTERSPACE RATIO  
POSITION OF TEETH  
IMMERSION  
CURVE NUMBER

70 CM  
5x15 CM PLATES  
5 CM  
STAGGERED  
16 CM  
2

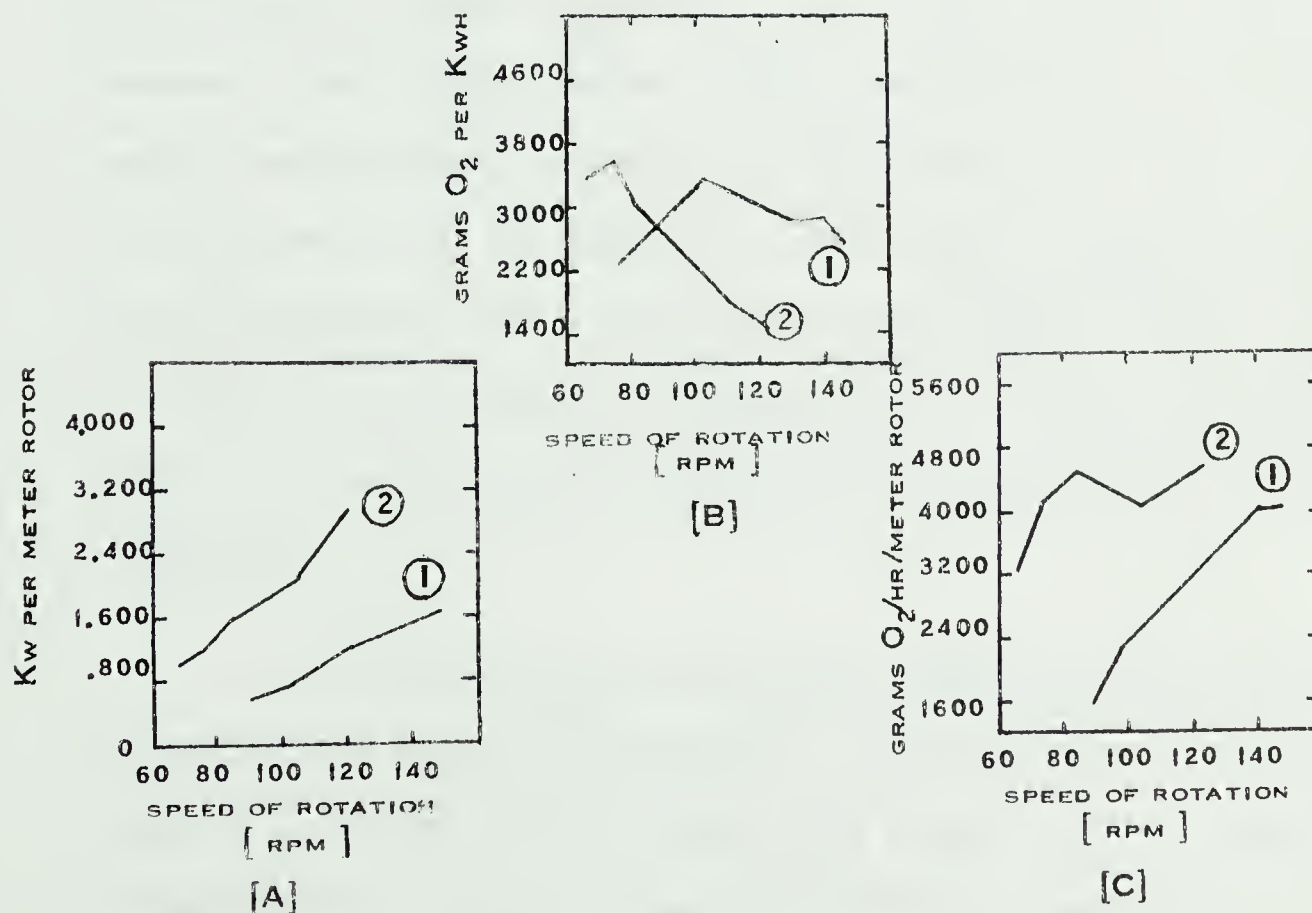


FIGURE : 7 ANGLE AND CAGE ROTOR PERFORMANCE COMPARISON

[ MODIFIED AFTER BAARS AND MUSKAT, 1959 ]



Pasveer's (1951) study of oxygen distribution in activated sludge floc shows that the oxygen penetrates the floc by diffusion and becomes bound in the floc. Further experiments confirmed theoretical reasoning that with a high oxygenation capacity device it is possible to increase the degree of turbulence and reduce the floc size, thereby maintaining a high concentration of oxygen in the water-floc interfacial surfaces and accelerating the process of biochemical oxidation. No adverse effects on the sludge properties due to the reduced floc size were reported. It was noted that during its passage from the aeration tank to the secondary settling tank the floc coagulated rapidly to form larger floc sizes. It could also be observed that when the brush is stopped for a brief period the liquid surface was seen to "break" as a result of sludge coagulation. As a result of pilot plant and laboratory experiments Pasveer (1951) concluded that by increasing the rate of oxygen supply together with the degree of turbulence the rate of purification can be increased to many times the rate found in many purification plants. From a practical point of view it may be said that in every activated sludge plant of today, the limiting factors for the purification capacity of the plant are turbulence and oxygenation.

In 1953 research by the Public Health Engineering Institute in Holland led to the development of a low-cost, high purification capacity pollution control system. The method consists chiefly of a single process; that of intense oxygenation of waste in an aeration



tank of the simplest possible construction. By intensive aeration over an extended period it is possible to oxidize a waste to such an extent that surplus sludge produced in the system and wasted periodically can be dried without giving off objectionable odors. In effect, because of the extended aeration period involved, the process is one of aerobic digestion. Pretreatment, sludge digestion and even a secondary settling tank and sludge return system may be deleted by selecting a suitable operating method.

Practise has indicated that in order to maintain a continuous process in larger installations a secondary settling tank is required. For an installation of this type the process components are as indicated in FIGURE 8. In the case of very small installations the oxidation ditch is also used as a settling tank. The settled supernatant is decanted by way of a siphon arrangement which is activated by discharging raw waste into the oxidation ditch. The raw waste is stored in a holding tank during the settling period. While excess sludge disposal from small plants is effected by way of a sludge trap located within the aeration tank, sludge from larger installations is wasted directly from the sludge pits within the secondary clarifier.

After the original development work was completed in 1957 the oxidation ditch has been readily accepted not only in the Netherlands, but also in other Western European countries. In





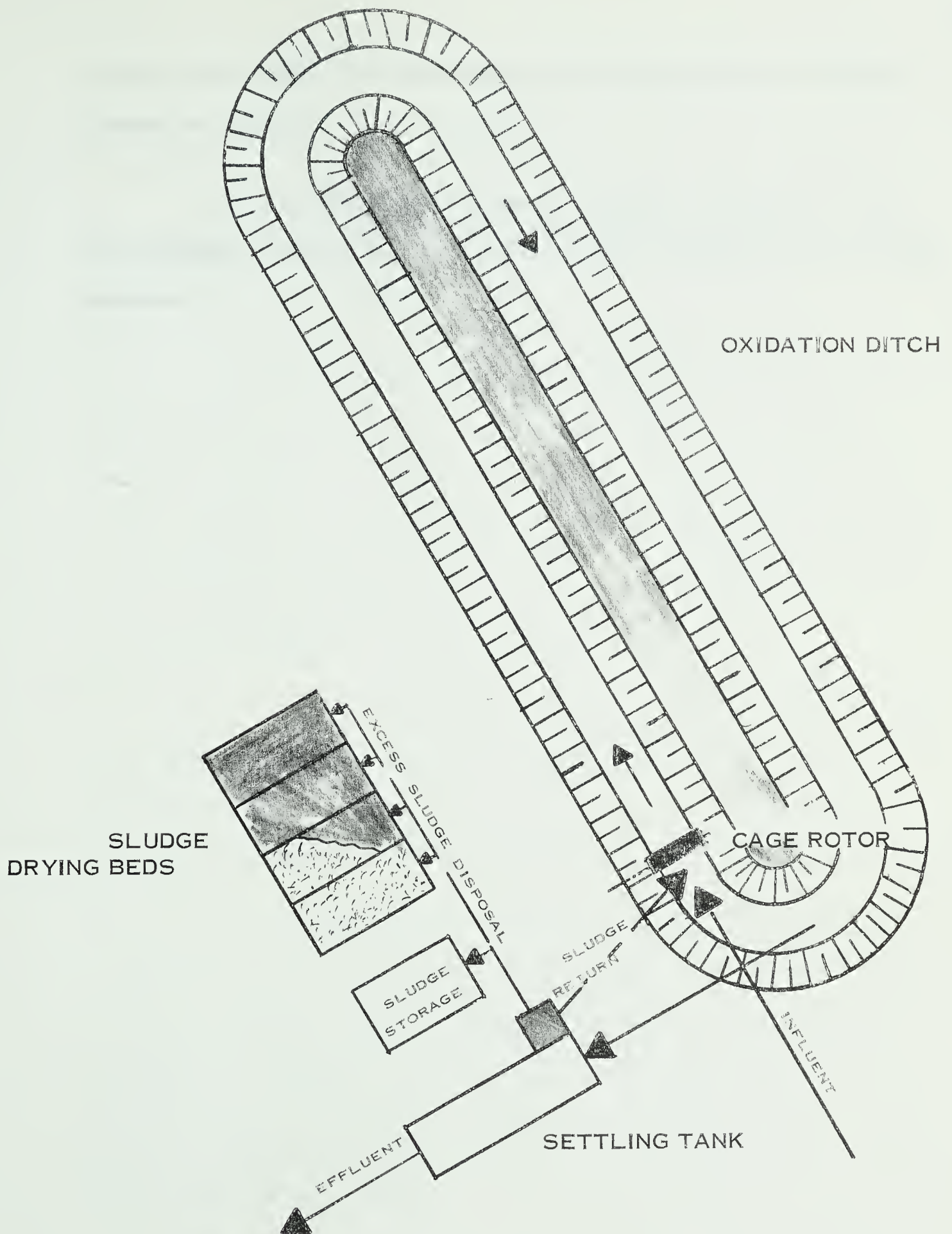


FIGURE : 8 OXIDATION DITCH PROCESS COMPONENTS



recent years (1962) the process has been introduced to the United States and Canada.

The oxidation ditch process consists of complete mixing and extended aeration and is a modification of the activated sludge process.



## CHAPTER III

### THEORY OF COMPLETE MIXING ACTIVATED SLUDGE

#### 1. Complete Mixing Activated Sludge

A complete mixing activated sludge system is characterized by immediate intermixing between influent and aerator contents to the extent that the influent particles lose their identity. The fluid in the vessel is completely mixed so that its properties are uniform and identical with those of the effluent. A mathematical treatise of complete mixing is given by Rich (1961).

In a conventional activated sludge system untreated wastes are mixed with return sludge at one end of the aeration tank and the micro-organisms receive the full impact of any shock load thereby responding with sudden increases in oxygen demand and in growth.

In a complete mixing activated sludge plant the large volume of the aeration tank functions also as a surge tank, reducing the effect of any sudden toxic waste discharge on the microbial population. The major advantage lies in that the food to organism ratio is constant throughout the tank, resulting in uniform oxygen demand and biologic growth.





Examination of the basic pattern of bacterial growth indicates that the rate of growth is either a function of the bacteria concentration or a function of the food concentration (FIGURE 9).

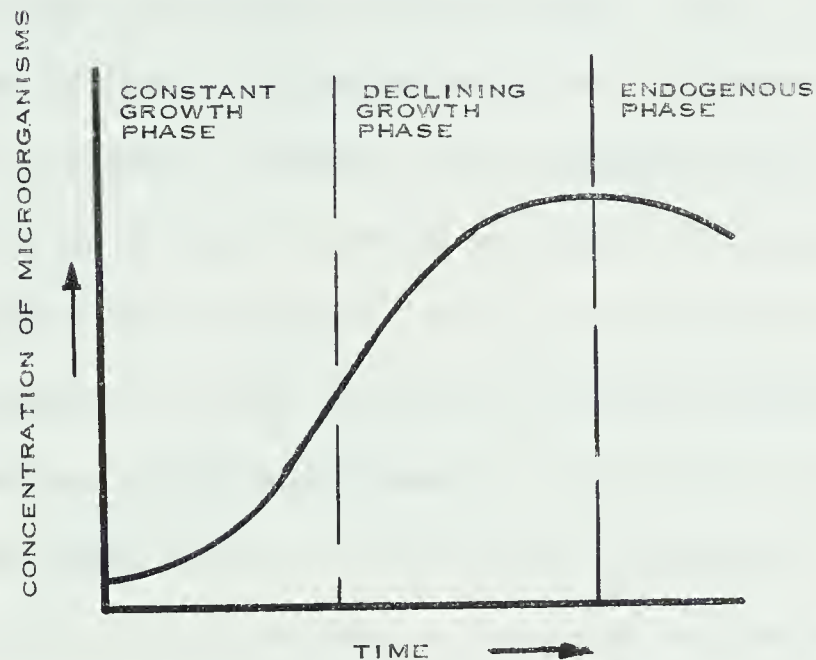


FIGURE : 9 CLASSIC GROWTH PATTERN

If the bacterial population is small and the food supply unlimited, the bacteria's growth rate is limited only by its ability to process organic matter. The rate of increase of bacterial mass continues in proportion to the mass of active bacteria until a point is reached when food becomes the limiting factor. This point has been designated as a food to organism ratio ( $F/M$ ) which varies between 2.1 and 2.5 (McKinney, 1962). McKinney shows that below the critical  $F/M$  ratio of 2.1 to 2.5 the rate of synthesis is a function of the remaining food concentration while above the critical  $F/M$  ratio the rate of synthesis is related to the mass of the active cells in the system.





Complete mixing systems are designed to operate in the declining growth phase of micro-organisms during which the rate of metabolism is a function of the concentration of the food remaining. Hence the food limiting situation controls the microbial growth in complete mixing activated sludge systems. This illustrates the importance of food to organism ratio as a parameter in aerobic waste treatment processes. Stewart (1964) defines the food to organism ratio ( $F/M$ ) by a factor  $F$  which represents the number of lbs. of BOD-5 applied per day per lb. of volatile suspended solids contained in the treatment system. Here it should be noted that Stewart's  $F$  is a measure of the total mass of the volatile suspended solids. As will be shown later the total mass of suspended volatile solids is made up of the active mass of bacteria and the mass formed by endogenous metabolism. FIGURE 10 shows a plot of process reaction rates against the relative organism weight designated by Stewart as  $1/F$ , and illustrates the high suspended solids concentration which is characteristic of extended aeration waste treatment systems.

Curve 1 indicates the amount of applied BOD-5 which is assimilated by processes operating at various relative organism weights. The organics are broken down to protoplasm and storage products by the micro-organisms and in part disappear from the system as a result of energy production during assimilative respiration. Curve 2 indicates the protoplasm and storage products as new growth VSS. The micro-organisms also require a small amount of energy to



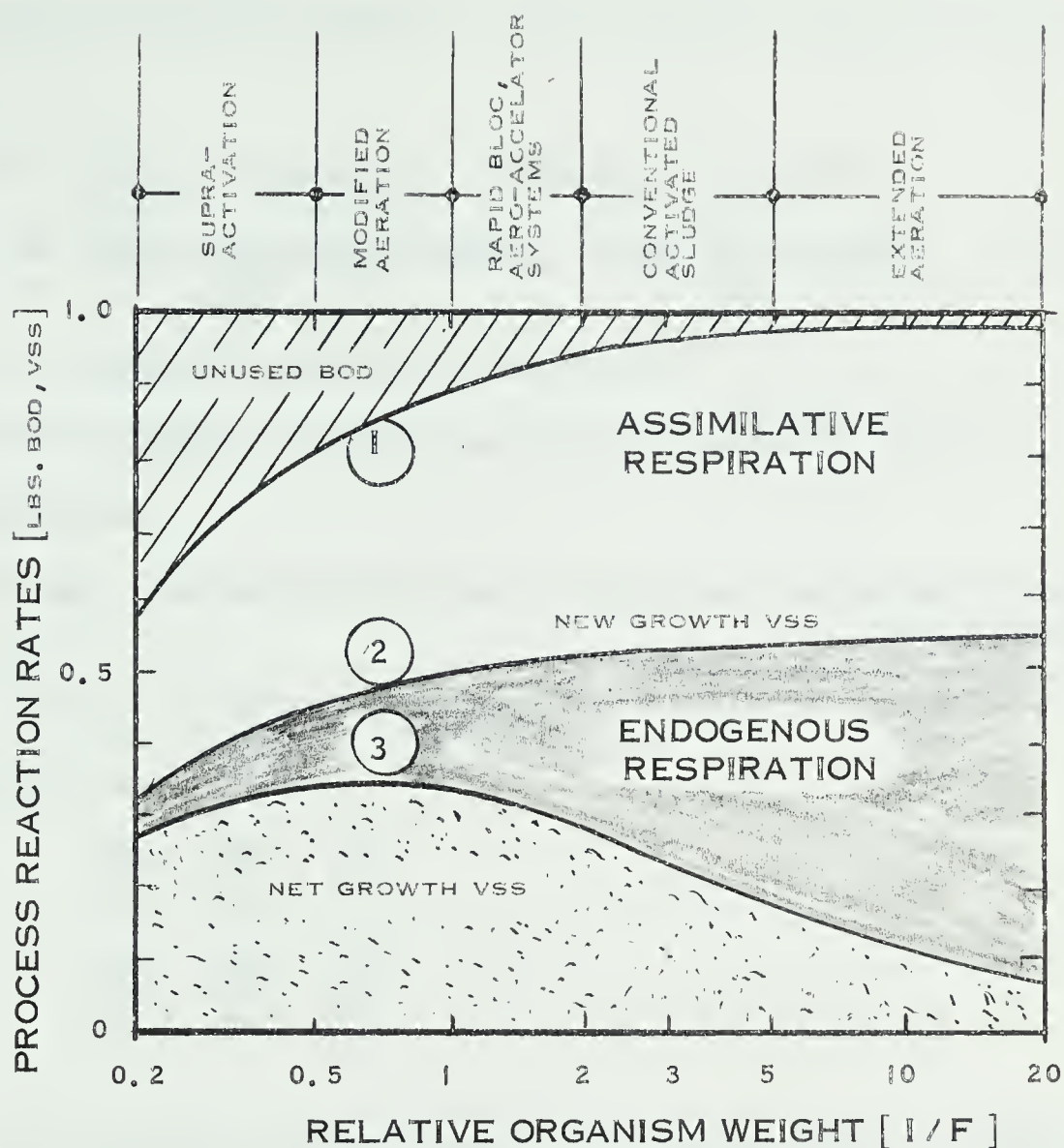


FIGURE : 10 PROCESS REACTION RATES vs. ORGANISM WEIGHT  
[ AFTER STEWART, 1964 ]

maintain normal functions (motion and enzyme activation). This basal energy requirement of the bacteria is designated as endogenous respiration. The remaining material designated by curve 3 is the new growth of organisms or may be termed the net protoplasm accumulation.



From FIGURE 10 a general growth Kinetics equation may be written:

Net Growth = New Growth - Endogenous Decrease

$$\frac{\text{lbs. Excess Volatile Sludge}}{\text{day}} = \frac{C(\text{lbs. BOD removed})}{\text{day}} - k(\text{lbs.V. sludge})$$

where: C = fraction appearing as new growth

k = endogenous decrease growth rate constant, per day

We can also write:

Organic Matter Metabolized = Protoplasm Synthesized + Energy  
for Synthesis.

McKinney (1962) defined the growth kinetics equations

mathematically as:

$$\frac{dF_m}{dt} = \frac{dM_s}{dt} + \frac{dO_s}{dt} \quad 1.$$

$$\frac{dM_p}{dt} = \frac{dM_s}{dt} - \frac{dO_e}{dt} \quad 2.$$

where:

F<sub>m</sub> = ultimate oxygen demand of the organic matter metabolized

M<sub>s</sub> = oxygen equivalent of the protoplasm synthesized

O<sub>s</sub> = oxygen uptake for synthesis

M<sub>p</sub> = net protoplasm accumulation as oxygen equivalent

O<sub>e</sub> = endogenous oxygen uptake

The direct relationship between synthesis and energy can be expressed by:

$$\frac{dO_s}{dt} = k_1 \frac{dM_s}{dt} \quad 3.$$

where:

k<sub>1</sub> = synthesis constant\*

\*see Appendix B - Table XII.124 for k-Factors





From the aforementioned equations a direct relationship between protoplasm synthesized and organic matter metabolized can be written as:

$$\frac{dF_m}{dt} = (1 + k_1) \frac{dM_s}{dt} \quad 4.$$

After death of the bacterial cell, the cell undergoes lysing with subsequent release of nutrients from the cell to the environment. The dead bacterial cell is inert organic matter not metabolizable by other bacteria and accumulates in proportion to endogenous metabolism. This may be mathematically expressed by:

$$\frac{dO_e}{dt} = k_2 Ma \quad 5.$$

where:

$k_2$  = endogenous respiration constant

$Ma$  = active or living mass of micro-organisms

These five equations make it now possible to interrelate the organic matter metabolized, growth and oxygen utilization.

Examination of FIGURE 10 very clearly reveals the advantage of operating a waste treatment process in the extended aeration range. The unused BOD is very low and so is the net VSS accumulation within the system. Other modified activated sludge systems operating over a range of relative organism weights are also illustrated in FIGURE 10; with decreasing relative organism weight both the unused BOD portion and the net VSS growth increase.



In complete mixing activated sludge systems three process variations are possible:

1. aeration only
2. excess sludge wasted in effluent
3. separate sludge wasting

The study carried out in connection with this theses deals with a system which uses separate sludge wasting. However, during the period of study no sludge was wasted inasmuch as only sludge return from the settling tank to the oxidation ditch was practiced. In effect then, a system of excess sludge wasting in the effluent would represent a closer description of the system under study.

## 2. Mathematical Model of Aeration Ditch

In any waste treatment process design it is imperative that a prediction of effluent characteristics be made. Effluents are generally described by two parameters, BOD-5 and Suspended Solids.

McKinney (1962) has presented mathematical analyses of the various complete mixing activated sludge processes. Using McKinney's terminology the Town of Olds Pollution Control Centre can be mathematically described by FIGURE 11.



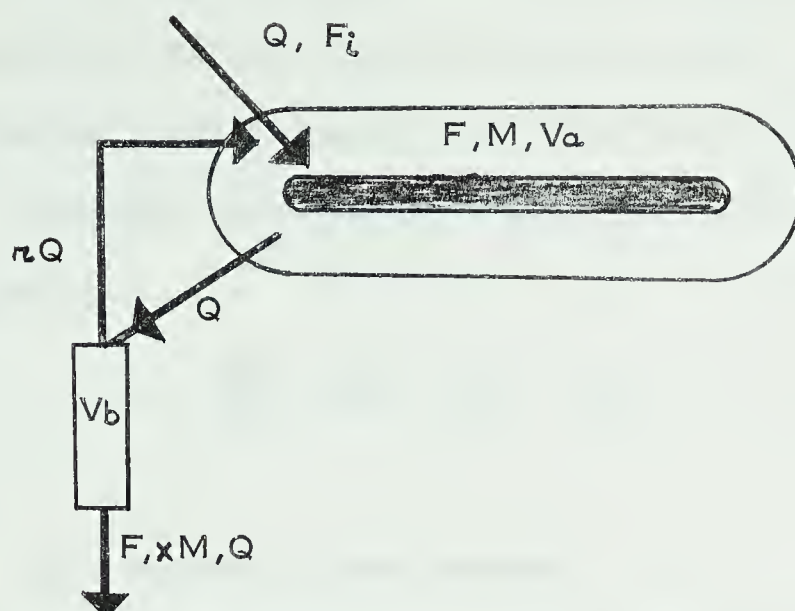


FIGURE : II MATHEMATICAL AERATION DITCH MODEL

where:

$F_i$  = organic wastes in influent

$Q$  = average inflow rate

$F$  = organic concentration in the oxidation ditch and effluent

$M$  = mixed liquor volatile suspended solids

$V_a$  = volume of oxidation ditch

$r$  = recirculation ratio

$V_b$  = volume of settling tank

$x$  = decimal fraction

$V$  = total volume of system ( $V_a + V_b$ )

$t$  = total detention time within the system



The organic concentration  $F$  in the system of volume  $V$  is increased by the addition of fresh wastes ( $F_i$ ), decreased by metabolism and displacement of the mixed liquor by the incoming wastes.

The rate of change of organic concentration can now be represented by:

$$\frac{VdF}{dt} = QF_i - QF - V k_5 F \quad 6.$$

where:

$k_5$  = metabolic rate constant

Since  $Q/V = 1/t$ , for equilibrium conditions when  $\frac{dF}{dt} = 0$ , equation 6 reduces to:

$$0 = \frac{F_i}{t} - \frac{F}{t} - k_5 F$$

$$F = \frac{F_i}{k_5 t + 1} \quad 7.$$

Hence, the effluent organics concentration ( $F$ ) when the organic concentration is limiting will be related to the incoming organic concentration ( $F_i$ ) and the system detention time ( $t$ ).

The total mass of volatile suspended solids,  $M$ , is the sum of the active mass,  $M_a$ , and the mass formed by endogenous metabolism,  $M_e$ .

$$M = M_a + M_e$$

The active mass of bacteria ( $M_a$ ) in the system may be calculated similarly to the rate of change of organic concentration. McKinney (1962) refers to observations that the bacteria in the raw wastes represent less than 1% of the bacteria in the system. Hence





the rate of change of bacteria in the system is equal to the bacteria synthesized less the endogenous respiration less the fraction of displaced bacteria. The rate of change of the active mass of bacteria (Ma) is given by:

$$\frac{VdMa}{dt} = Vk_6F - Vk_7Ma - QxMa \quad 8.$$

For equilibrium conditions  $\frac{dMa}{dt} = 0$  and with  $\frac{Q}{V} = 1/t$  equation 8 reduces to:

$$0 = k_6F - k_7Ma - \frac{xMa}{t}$$

$$Ma = \frac{k_6F}{\frac{x}{t} + k_7} \quad 9.$$

where:

$k_6$  = synthesis constant

$k_7$  = endogenous respiration constant  
(related to  $k_2$ )

Equation 9 shows that the active bacterial mass in the system is a function of the total residence time within the system and the organic load on the system.

Because the endogenous mass,  $Me$ , is a direct function of the active mass,  $Ma$ , the total mass of volatile suspended solids is related to the active mass. The change in the rate of build-up of endogenous mass in the system is equal to the endogenous respiration less the fraction of displaced solids formed by endogenous metabolism.

$$\frac{VdMe}{dt} = Vk_8Ma - QxMe \quad 10.$$



For equilibrium conditions  $dMe/dt = 0$ , and with  $Q/V = 1/t$ , equation 10 is reduced to:

$$\begin{aligned} \frac{1}{t}xMe &= k_8 Ma \\ Me &= \frac{k_8 Mat}{x} \end{aligned} \quad 11.$$

where:

$k_8$  = endogenous metabolism constant

Equation 11 indicates that the endogenous mass,  $Me$ , within the system is a direct function of the sludge fraction ( $x$ ) lost in the effluent. The loss of metabolizable solids in the effluent is generally assumed to be zero. This is due to the greater time available for the metabolism of metabolizable solids within the system. The major difficulty with a sludge recirculation system is the determination of the value of  $x$ , which is related to the maximum solids concentration that can be carried in the aeration tank and to the inert, volatile solids in the effluent. The maximum solids concentration ( $M_t$ ), that can be carried in the oxidation ditch is related to only two parameters, the sludge volume-index (SVI) and the sludge return rate ( $R$ ). A mass balance of the suspended solids contained in an aeration tank yields the following equation:

$$M_t = \frac{10^6 \left( \frac{R}{Q} \right)}{\left( 1 + \frac{R}{Q} \right) \text{SVI}} \quad 12.$$

When plotting equation 12 for various values of SVI and  $R/Q$  (FIGURE 12) several important aspects of aerobic treatment plant operation become evident. When the SVI is relatively high



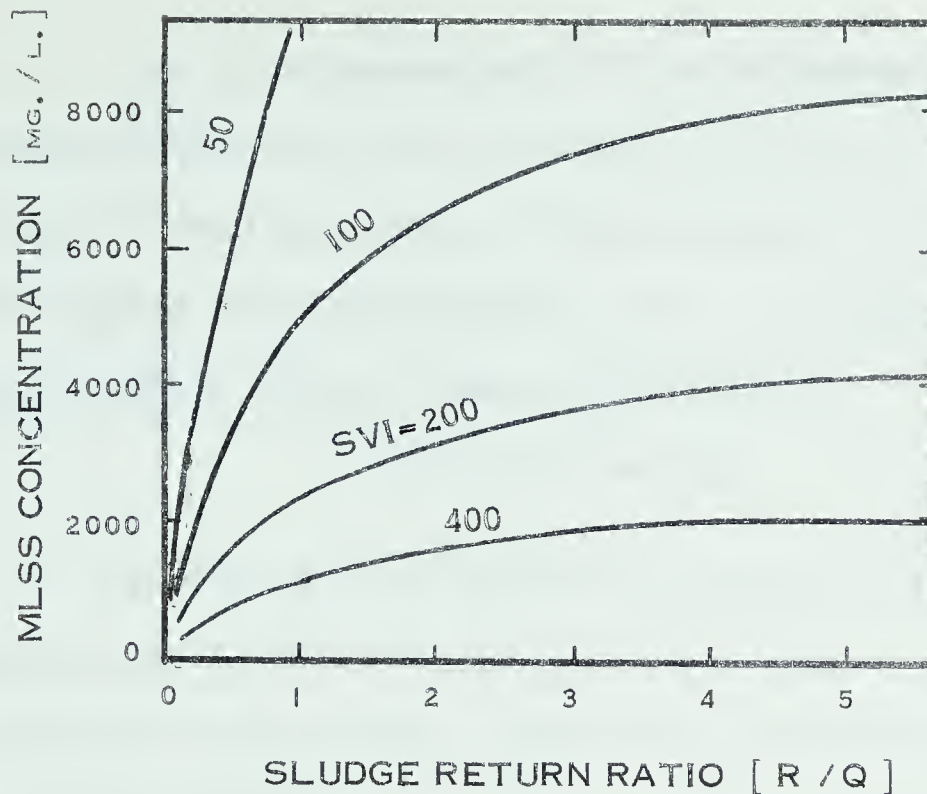


FIGURE : 12 MLSS CONCENTRATION vs. SLUDGE RETURN RATIO

(greater than 100) it becomes very difficult to maintain a high suspended solids concentration in the mixed liquor even though the return sludge rate is great. In order to attain the desirable operating mixed liquor suspended solids concentration (MLSS) of 6000 mg/l in an oxidation ditch, a SVI of 100 or less is required.

The inert volatile solids  $(M_i)_i$  in the raw wastes will build up in the aeration system until equilibrium conditions are attained. At that point the inert volatile solids in both influent and effluent will be equal.

$$(M_i)_i = xM_i$$

13.

where:

$M_i$  = inert volatile suspended solids in the mixed liquor.





The total concentration of volatile suspended solids,  $M$ , in the oxidation ditch will be equal to the sum of the active mass synthesized,  $M_a$ , the mass formed by endogenous metabolism,  $M_e$ , the metabolizable volatile suspended solids,  $M_m$ , and the raw waste unmetabolizable volatile suspended solids,  $M_i$ .

$$M = M_a + M_e + M_m + M_i \quad 14.$$

By substituting expressions for  $M_e$  and  $M_i$  as determined in equations 11 and 13 and assuming that  $M_m$  is zero, equation 14 can be rewritten in terms of  $x$ , the fraction of excess activated sludge lost in the effluent:

$$M = M_a(1 + \frac{k_8 t}{x}) + \frac{(M_i)i}{x}$$

Solving for  $x$ ,

$$x = \frac{M a k_8 t + (M_i)i}{M - M_a} \quad 15.$$

With the value of  $x$  it is possible to determine the quantity and character of the excess activated sludge lost in the effluent.

Since the rate of oxygen utilization in the oxidation ditch is related to synthesis and endogenous respiration it can be defined by:

$$\frac{dO}{dt} = k_9 F + k_2 M_a \quad 16.$$

where:

$k_9$  = oxygenation constant

$k_2$  = endogenous metabolism constant



at equilibrium  $\frac{dO}{dt} = 0$  and the total oxygen demand, (T.O.) can be calculated from:

$$T.O. = (k_9 F + k_2 Ma)t \quad 17.$$

Equation 17 shows that the total oxygen demand of the system is related to the organic concentration,  $F$ , and the active microbial mass,  $Ma$ . The effluent oxygen demand BOD-5 can then be expressed by:

$$\text{Effluent BOD-5} = F + k_{10} xMa \quad 18.$$

where:

$F$  = effluent organic matter in terms of BOD-5

$k_{10}$  = carbonaceous metabolism constant at 20 degrees Centigrade over a 5-day period

$xMa$  = fraction of active solids in the effluent

Of the equations presented, four are of direct interest to the practical engineer:

1. Equation 12, which indicates the maximum solids concentration which can be carried in the system.

$$M_t = \frac{10^6 \left(\frac{R}{Q}\right)}{\left(1 + \frac{R}{Q}\right) SVI}$$

2. Equation 16, predicting the rate of oxygen utilization of the process.

$$\frac{dO}{dt} = k_9 F + k_2 Ma$$

3. Equation 18, predicting the effluent BOD-5 of the process.

$$\text{Effluent BOD-5} = F + k_{10} xMa$$



4. From the various mathematical relationships presented earlier it is possible to predict the effluent volatile suspended solids (VSS) concentration.

$$\text{Effluent VSS} = xM$$

Provided the biodegradability of a waste to be treated has been ascertained, the equations just presented in conjunction with established treatment plant effluent requirements will make it possible to embark on a successful design of a complete mixing activated sludge treatment system.



## CHAPTER IV

### SIGNIFICANCE OF SOME POLLUTION CONTROL PARAMETERS

Before describing the field studies conducted during the testing program, a brief review of the significance of some of the pollution control parameters such as 5-day biochemical oxygen demand, chemical oxygen demand, volatile suspended solids, phosphates, nitrogenous compounds, coliform concentrations and dissolved oxygen levels is in order.

#### 1. Biochemical Oxygen Demand

The biochemical oxygen demand (BOD) determination is a measure of the amount of oxygen required to oxidize the organic matter in a sample by way of microscopic organisms. The test consists of determining the dissolved oxygen concentration in a sample prior to and following a 5-day incubation period at 20 degrees Centigrade. The amount of oxygen used during the 5-day incubation period is known as the 5-day biochemical oxygen demand (BOD-5).

Since raw sewage exerts an oxygen demand and all waste treatment processes are designed to satisfy this demand, the percent reduction in BOD-5 is generally used as a parameter in evaluating waste treatment process efficiency. McKinney (1962) advises caution in using BOD-5 as a parameter in treatment plant efficiency evaluation of complete-mixing extended aeration activated sludge systems. He notes





that the oxygen demand of the raw waste is caused by the metabolism of the raw waste by the micro-organisms and the oxygen demand of the completely mixed activated sludge effluent is caused by the endogenous metabolism of the raw wastes. Hence the same thing is not measured and considerable error and unnecessary engineering problems could result.

BOD-5 determinations were made in accordance with procedures outlined in the 12th Edition of Standard Methods. (1965)

## 2. Chemical Oxygen Demand

The chemical oxygen demand (COD) test is widely used as a means of measuring the pollutorial strength of sewages and industrial wastes. It is based on the fact that nearly all organic compounds can be oxidized to carbon dioxide and water by the action of strong oxidizing agents under acid conditions regardless of the biological assimilability of the substances. As a result, COD values are greater than BOD values and may be much greater when significant amounts of biologically resistant organic matter is present.

One of the chief limitations of the COD test is its inability to differentiate between biologically oxidizable and biologically inert organic matter. In addition, it does not provide any evidence of the rate at which the biologically active material would be stabilized under conditions that exist in nature.



The major advantage of the COD test is the short time required for evaluation. The determination can be made in about three hours rather than the five days required for the measurement of BOD. For this reason it is used as a substitute for the BOD test in many instances. COD data can often be interpreted in terms of BOD values after sufficient experience has been accumulated to establish reliable correlation factors.

However, since BOD involves metabolic processes which are temperature dependent and COD involves chemical processes which are not as temperature sensitive, COD/BOD correlation factors should differ for different temperature ranges.

COD determinations were made in accordance with procedures outlined in the 12th Edition of Standard Methods.

### 3. Volatile Suspended Solids

One of the major objectives of performing solids determinations upon sewages, industrial wastes and sludge samples is to obtain a measure of the amount of organic matter present. This test is accomplished by a combustion procedure after filtration in which the organic matter is converted to carbon dioxide and water. To prevent decomposition and volatilization of inorganic substances as much as is consistent with complete oxidation of the organic matter, ignitions are conducted at 600 degrees Centigrade. The loss in weight is interpreted in terms of organic matter representing the total mass of volatile suspended



solids which, as noted in CHAPTER III is made up of an active and an inactive biological mass. The active mass could be determined either polarographically or by using the Warburg Respirometer.

Volatile Suspended Solids determinations were carried out using Gooch crucibles in accordance with procedures outlined in the 12th Edition of Standard Methods.

#### 4. Phosphates

Data regarding phosphorus concentrations is increasing in significance as engineers have become cognizant of its vital role in life processes. It is also important in the assessment of potential biological productivity of surface waters.

Phosphorus is present in two major forms. Organically bound phosphorus, which is of minor consequence and inorganic compounds of phosphorus which is of major concern. The inorganic compounds of phosphorus appear as phosphates or their molecularly dehydrated forms usually referred to as polyphosphates.

Prior to the development of synthetic detergents inorganic phosphorus concentrations in wastes usually ranged from 2 to 3 mg/l. This was chiefly attributed to human wastes as a result of the metabolic breakdown of proteins and elimination of the liberated phosphates in urine. Hence the amount of phosphorus released is a function of protein intake.

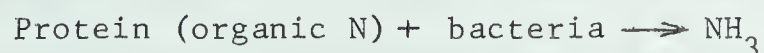




Phosphate determinations were made in accordance with the aminonaphtholsulfonic acid method as outlined in the 12th Edition of Standard Methods.

## 5. Nitrogenous Compounds

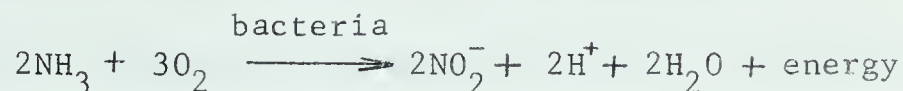
The compounds of nitrogen are of great interest to sanitary engineers because of the importance of nitrogen in the life processes of all plants and animals. In all biological waste treatment systems, it is necessary that the micro-organisms have all the necessary elements to form protoplasm. Domestic sewage contains all the elements that the bacteria require but some industrial wastes are deficient in key elements. Nitrogen and phosphorus to name two. One of the most important aspects of nutritionally deficient wastes is their effect on biological predomination. For example, a partially nitrogen-deficient waste will stimulate fungi over the bacteria since the fungi form protoplasm with a lower nitrogen content than bacteria. The fungi which would then predominate are filamentous and prevent good settling. The feces of animals contain appreciable amounts of unassimilated protein matter (organic nitrogen). It and the protein matter remaining in the bodies of dead animals and plants are converted to ammonia by the action of saprophytic bacteria under either aerobic or anaerobic conditions.



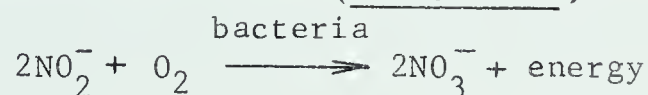
The ammonia released by bacterial action on urea and proteins may be used by plants directly to produce plant protein. If it is



released in excess of plant requirements, the excess is oxidized by nitrifying bacteria. One group, known as the nitrite formers (Nitrosomonas), convert ammonia under aerobic conditions to nitrites and derive energy from the oxidation.



The nitrites are oxidized by a second group of nitrifying bacteria called the nitrate formers (Nitrobacter).



The nitrates formed may serve as fertilizer for plants.

In view of recent emphasis on the eutrophication of effluent receiving bodies of water, the control of nitrates and phosphates in sewage effluents is receiving more attention.

All nitrogenous compounds were analysed in accordance with procedures outlined in the 12th Edition of Standard Methods. More specifically ammonia and nitrate determinations were made using the direct nesslerization and phenoldisulfonic acid methods respectively.

## 6. Coliform Organisms

Coliform organisms have been extensively employed as indicators of possible fecal contamination of water. It is therefore of interest to discharge effluent low in coliform concentration specifically if the receiving stream constitutes a water supply source for others. The bactericidal effect of various waste-water



treatment systems on coliform organisms has been reported in the literature. It has been found that the environment established in an activated sludge system is such that a marked reduction in coliform population occurs, but the reason for the rapid destruction of coliform bacteria is not clearly understood and many theories and mechanisms responsible for the destruction have been postulated; production of materials toxic to coliform bacteria and competition for nutrients to name two.

## 7. Dissolved Oxygen

All living organisms are dependent upon oxygen in one form or another to maintain the metabolic processes that produce energy for growth and reproduction.

The solubility of atmospheric oxygen in fresh water ranges from 14.6 mg/l at 0 degrees Centigrade to about 7 mg/l at 35 degrees Centigrade under 1 atmosphere of pressure. Since oxygen is a poorly soluble gas, its solubility varies directly with the atmospheric pressure at any given temperature. Because rates of biological oxidation increase with temperature and oxygen demand increases accordingly, high temperature conditions where dissolved oxygen is least soluble are of greatest concern. In aerobic biological treatment systems, the limited solubility of oxygen is of great importance because it governs the rate at which oxygen will be absorbed by the medium.

Since dissolved oxygen in liquid wastes determines whether



biological changes are brought about by aerobic or anerobic organisms, dissolved oxygen measurements are vital for maintaining aerobic conditions in aerobic treatment systems. The dissolved oxygen measurements are also indispensable as a means of controlling the rate of aeration to make certain that adequate amounts of air are supplied to maintain aerobic conditions and to prevent excessive use of air.

Copper Sulfate - Sulfamic Acid Flocculation Modification followed by the Azide Modification as outlined in the 12th Edition of Standard Methods was used.





## CHAPTER V

### FIELD STUDIES AND OBSERVATIONS

#### 1. Description of Pollution Control Centre

The oxidation ditch studies were conducted at the Town of Olds Pollution Control Centre, located approximately 145 miles south of Edmonton, Alberta on Highway No. 2 and one mile west of the Town Limits.

The installation as shown in FIGURE 13 serves a community of approximately 3000 persons and treats domestic wastes which are readily bio-degradable.

The incoming raw waste is discharged directly without pretreatment into the continuous earthen ditch. While experience in some European installations has indicated that neither bar screen nor grit removal facilities are required, the installation of a bar screen will be a definite requirement since the continual clogging of sludge return pumps represents a significant operational deficiency. Once the raw waste is in the ditch, it is violently agitated by two cage rotors. The function of the rotors is two-fold. It entrains oxygen in the waste and imparts a horizontal velocity component to the liquid. Oxygen is continually entrained as the waste is propelled around the ditch and repasses the rotor. A gravity pipe line allows



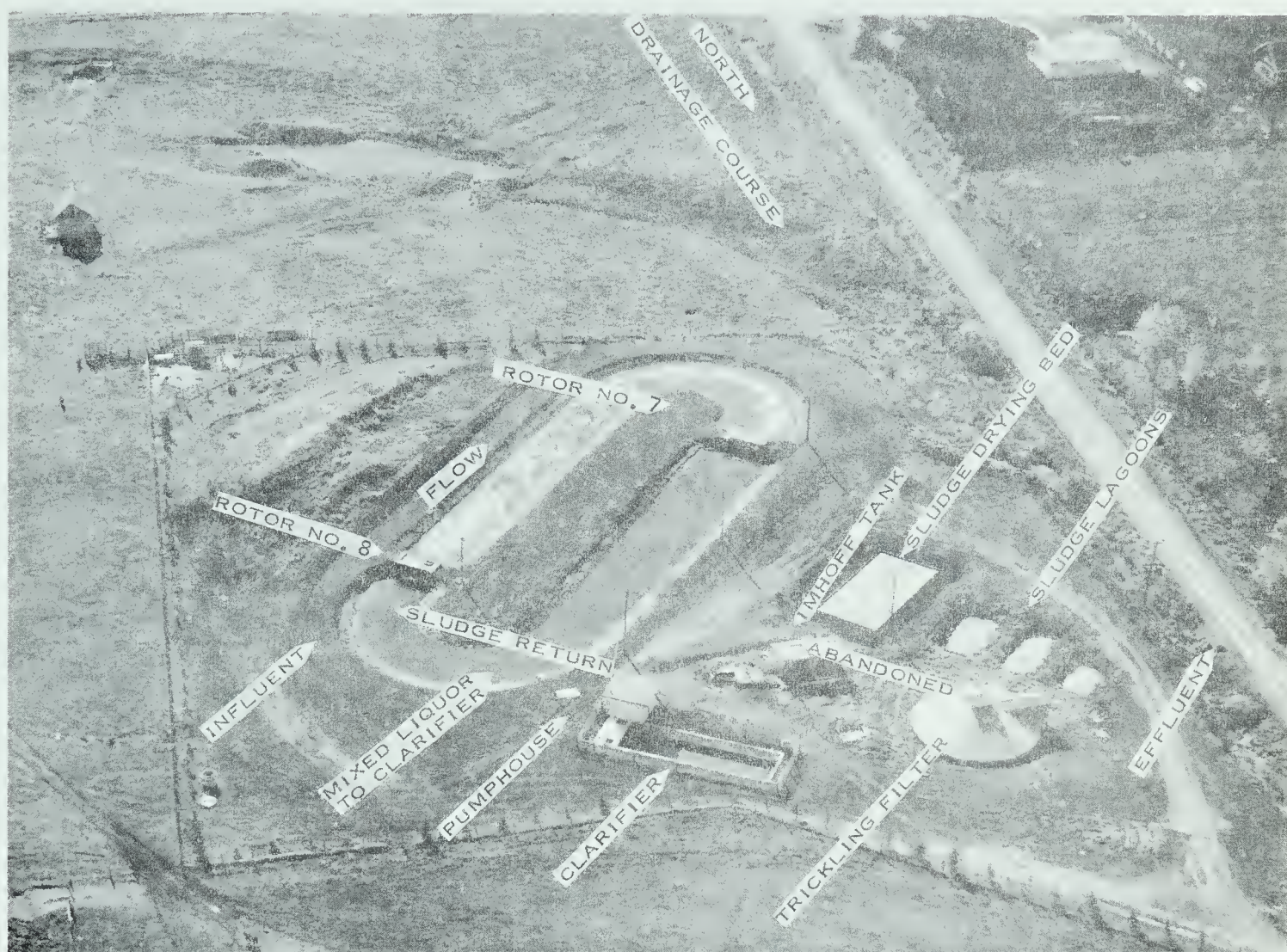


FIGURE : 13 TOWN OF OLDS POLLUTION CONTROL CENTRE





the mixed liquor to be continuously drawn off to a mechanized clarifier where the activated sludge is settled and immediately pumped back to the ditch. The clarified effluent is discharged to the receiving stream while excess sludge is either wasted to the drying bed or stored in the sludge lagoons. Liquid levels in the ditch are regulated by raising or lowering the effluent weir in the clarifier. Additional flexibility in liquid level control is possible by raising or lowering the complete rotor mechanism within each rotor structure in increments of six inches from a minimum of 4'-0" to 5'-0" liquid depth.

A detailed geometric description of the various process components for the Town of Olds Pollution Control Centre is found in Appendix B. .

## 2. Scope of Testing Program

One of the objectives of this thesis was to assess the efficiency and physical operation of the oxidation ditch in light of severe winter conditions. For this purpose the testing program extending from October 1966 to February 1967 was divided into three separate phases:

- Phase I    - Intermittent Testing Program  
              (October 1966 to November 1966)
- Phase II   - 4-day Continuous Testing Program  
              (December 28 - 31st, 1966)
- Phase III - Extended Testing Program  
              (January 3rd - February 20th, 1967)

Various analyses as summarized in Table I were carried out.





TABLE I

## TESTING PROGRAM OUTLINE

Test	Phase I		Phase II			Phase III	
	Eff.	ML	Inf.	Eff.	ML	Eff.	
BOD-5	*		*	*	*	*	
COD	*		*	*	*	*	
TSS	*		*	*	*	*	
VSS	*		*	*	*	*	
TS	*		*	*	*	*	
TVS	*		*	*	*	*	
PO <sub>4</sub> <sup>=</sup>			*	*	*	*	
NH <sub>3</sub> -N			*	*	*	*	
NO <sub>2</sub> -N			*	*	*		
NO <sub>3</sub> -N			*	*	*		
DO	*			*	*		
Temperatures			*	*	*		
Settleable Solids		*				*	



### 3. PHASE I - Intermittent Testing Program

Phase I of the project consisted of becoming acquainted with the installation and obtaining an appreciation of various treatment parameters. During this period only one rotor was operating continuously since the second rotor was being repaired. The period of single rotor operation was extensive and points up that standby equipment to maintain the process functioning is a necessary requirement. During this testing period which extended from October 10th, 1966 to November 29th, 1966 the average ambient temperature was +23.7 degrees Fahrenheit. The sampling consisted of obtaining grab samples of both effluent and mixed liquor. As indicated in TABLE II the average effluent BOD-5 was 32 mg/l. while the effluent COD averaged 216 mg/l. The average suspended solids in the effluent was 95 mg/l of which 73 mg/l were the average volatile portion.

With a sludge return ratio of 0.48 the sludge volume index (SVI) increased from 29 to 48 during Phase I of the testing program. As noted in CHAPTER III, FIGURE 12 the SVI is well below 100 and therefore is favorable to a continued increase in the MLSS concentration until the desirable process working range of 6000 mg/l MLSS is attained.

The effluent dissolved oxygen concentration approximated 0.4 mg/l with single rotor operation. Mixed liquor dissolved oxygen profiles for single rotor operation are shown in FIGURE 40.



TABLE II

## INTERMITTENT TESTING PROGRAM SUMMARY

## PHASE I

Date	EFFLUENT				MIXED LIQUOR			EFFLUENT
	BOD-5	COD	TSS	VSS	MLSS	SETTLEABLE	SVI	DO
	mg/l	mg/l	mg/l	mg/l	mg/l	SOLIDS ml	-	mg/l
Oct. 20/66	26	221	81	68	1076	31	29	-
Oct. 26/66	38	232	92	86	1500	50	33	0.2
Oct. 27/66	34	196	88	76	1456	50	34	0.5
Nov. 10/66	32	240	98	49	2508	120	48	0.5
Nov. 29/66	29	192	114	86	2248	105	48	-
Average	32	216	95	73				

## 4. PHASE II - 4-Day Continuous Testing Program

It was originally intended that a 4-day continuous testing program be started in early December. However, a two-week period of below zero temperatures lead to an ice formation on the clarifier surface which after becoming sufficiently thick to interfere with the sludge removal mechanism caused several wooden flights to break thereby making the settling tank inoperative. Since it was too cold to make immediate repairs, the complete treatment installation was taken out of operation. During this period raw sewage bypassed the treatment plant. After repairs were made the plant was started



up again on December 20th, 1966. To forestall reoccurrence of a further settling tank failure, the tank was covered with 2 x 12 planks and a polythene liner. No further freezing problems were encountered.

FIGURES 14 to 18 show operating conditions during the 4-day testing period under average ambient temperatures of +27 degrees Fahrenheit. As indicated in FIGURES 14, 15, 17 and 18 a layer of frozen foam covered the ditch with the exception of the straight stretches after each rotor. It was noted that the foam was most abundant in the early morning hours and had completely disappeared by mid-afternoon. The foam build-up can be partially attributed to the low MLSS concentration, which during Phase II was approximately 2260 mg/l. Later observations when the MLSS concentration had increased to approximately 4530 mg/l (March 22nd, 1967) revealed no further foaming.

Because of the extended detention time of the liquid in the ditch at low temperature, freezing of the ditch surface was anticipated. Possible ice formation on the ditch was not considered to be detrimental to the plant operation but rather beneficial in reducing heat losses from the mixed liquor. But since ice formation at the rotors could hinder the successful mechanical operation of the rotor, protective shrouds on both sides of each rotor structure were incorporated in the design (FIGURE 14).





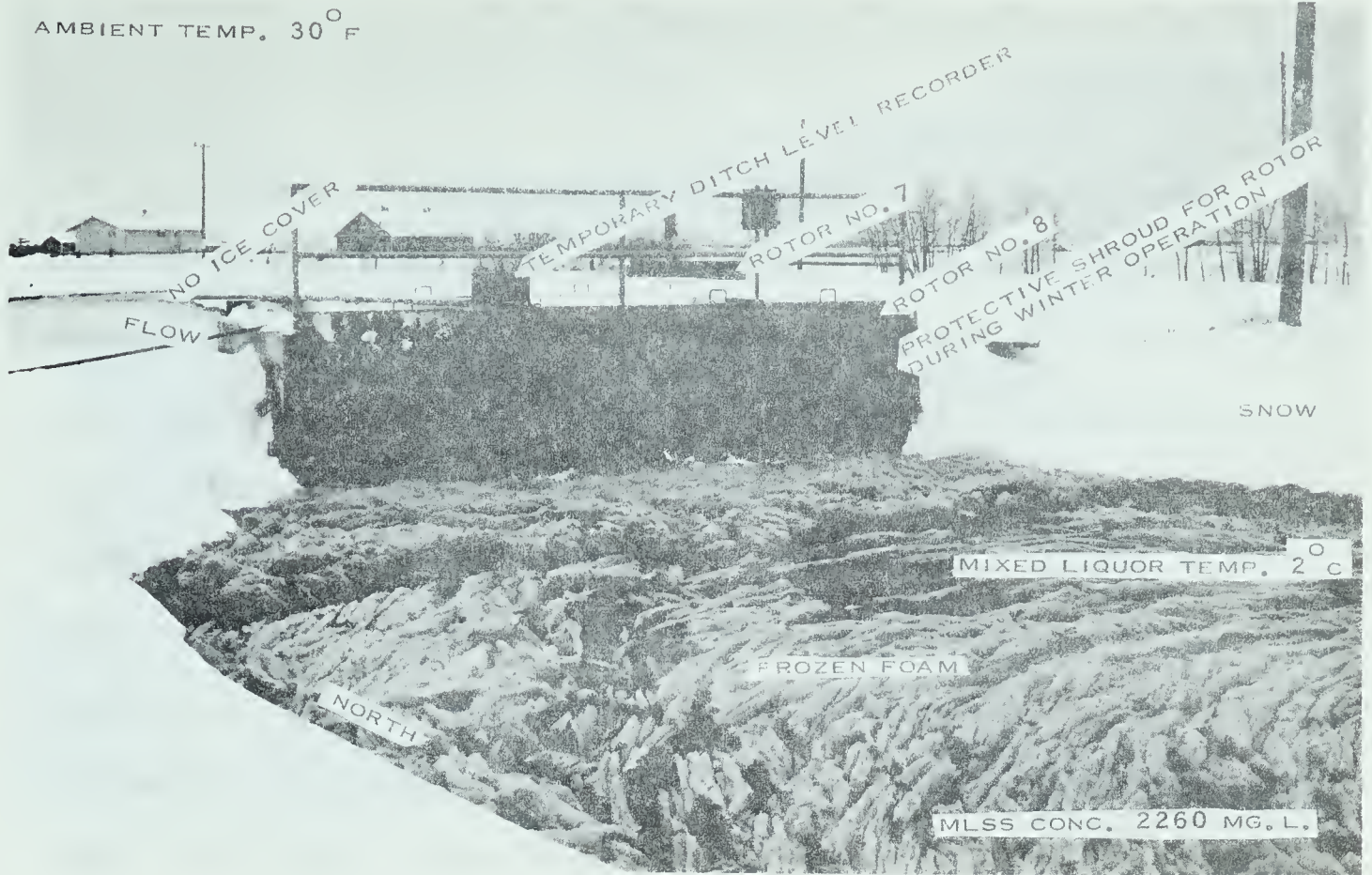


FIGURE : 14

OXIDATION DITCH WINTER OPERATION

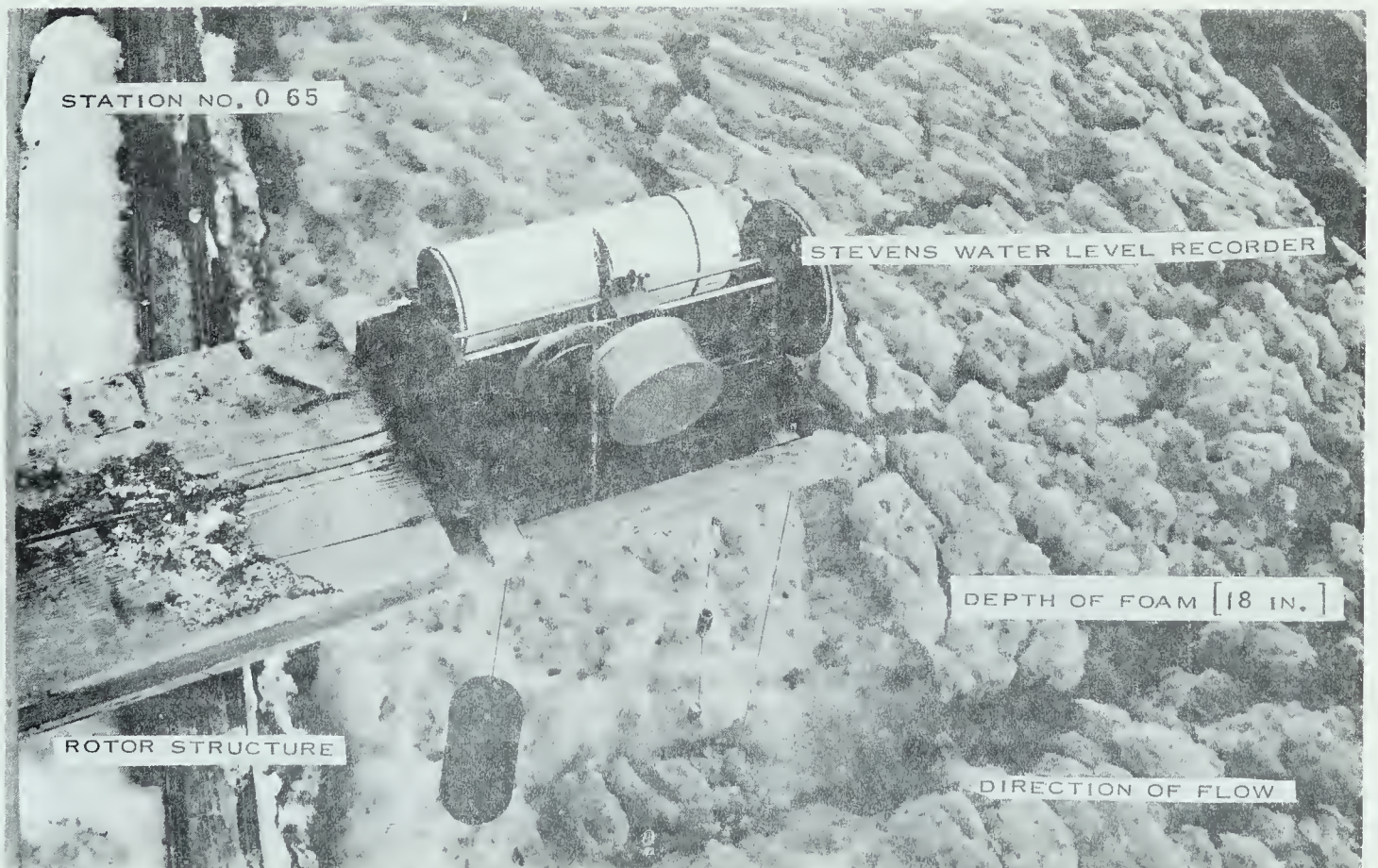


FIGURE : 15

DITCH LEVEL MONITORING





FIGURE 15 depicts a Stevens water level recorder which was used to determine liquid level variations during the day. After correcting for the effect of level fluctuations due to back-water waves near the rotor and the float line being out of plumb due to surface currents, the variation in sewage inflow caused the ditch level to fluctuate 0.66 feet during the day. This fluctuation leads to deeper immersion of the rotor and subsequent higher oxygenation capacity and power requirements at higher inflows. Unless the maximum rotor immersion is limited to safeguard rotor overloading large liquid level fluctuations are not desirable. In order to keep ditch level fluctuations to a minimum the conduit from the ditch to the clarifier should be of sufficient size to minimize increased head-loss at increased flows.

Evidence of scour immediately beyond each rotor structure is shown in FIGURE 16. Basic hydraulic design considerations for expanded channel sections dictate a more gradual transition section than used. Alternatively the whole rotor structure could be set on the channel banks and the rotor structure suspended from it thereby retaining the unrestricted channel cross-section. Rip-rap in the rotor vicinity would still be required because of the high degree of turbulence generated.

FIGURE 17 shows the formation of a sludge bar. This bar consists of an accretion of sludge deposit attached to the inside bend of the channel immediately upstream of Rotor No. 7. In the







FIGURE : 16 SCOUR NEAR ROTOR STRUCTURE



FIGURE : 17

SLUDGE BAR





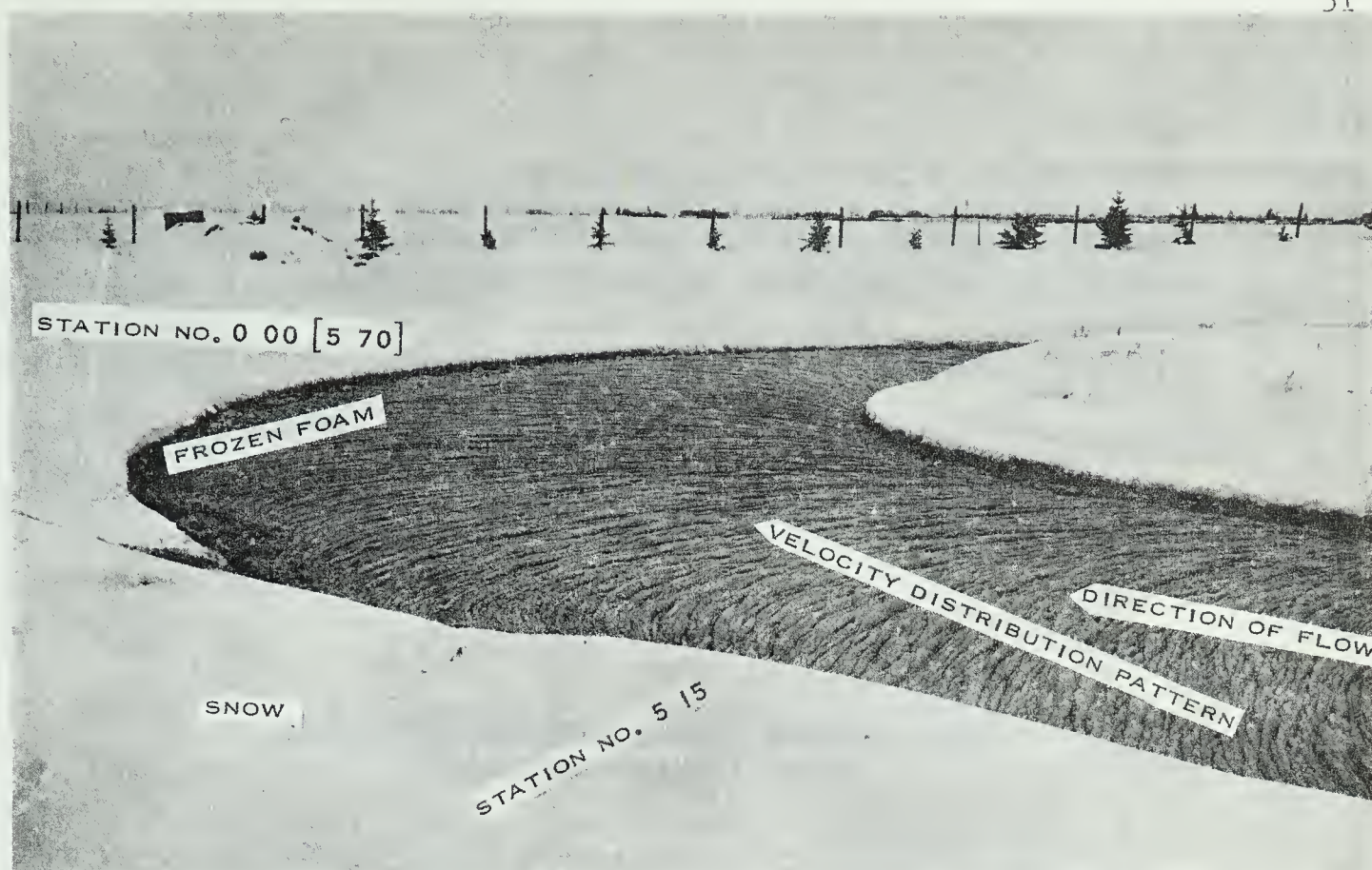


FIGURE : 18

## FOAM BUILD-UP

bend the axial velocity distribution is modified and maximum velocity crosses from near the inner bank at the start of the bend to the outer bank. The liquid actually screws itself around the bend thereby setting up a transverse circulation pattern. The velocity distribution pattern is well illustrated by the frozen foam as shown in FIGURE 18. The persisting lower velocity in the inside of the bend allows for deposition of the heavier sludge particles. Vertical guide vanes set into the channel and extending to within  $0.25d$  of the liquid surface would maintain a constant velocity around the inside of the bend and prevent the mixed liquor solids from depositing.

In order to establish average loading conditions for the treatment plant 4-hour composite samples of the influent were taken.



During this period flow measurements taken with a V-notch weir which was installed at the end of the settling tank indicated an average daily flow of 150,000 Imperial gallons. The results of laboratory analyses and calculated treatment plant loadings are tabulated in TABLE III.

TABLE III

COMPOSITE INFLUENT SAMPLE ANALYSIS AND  
TREATMENT PLANT LOADING

Description	Sample No.			Ave. mg/l	Flow Ave. lgpm	Load Ave. lbs./day
	16 mg/l	31 mg/l	42 mg/l			
BOD-5	307	252	334	298	104	446
COD	673	758	680	704		1054
TS	1695	1538	1677	1637		2455
VS	388	316	406	370		555
FS	1307	1222	1271	1266		1900
TSS	290	212	252	251		376
VSS	230	172	224	208		314
FSS	60	40	28	43		64.5
NH <sub>3</sub> -N	26	35	42	34		51
NO <sub>2</sub> -N	Tr	Tr	Tr	Tr		-
NO <sub>3</sub> -N	0.80	0.80	0.75	0.78		1.17
Total-N	53	58	68	60		90
PO <sub>4</sub>	5.8	3.0	5.2	4.7		7.5
Cl <sup>-</sup>	125	135	130	130		195



As indicated by the analyses the per capita BOD-5 contribution to the treatment plant is 0.15 lbs. per day at a flow of 50 Imperial gallons per capita per day (I.gpcd). This compares to average North American BOD-5 design figures of 0.17 lbs. per capita per day (lbpcd) and hydraulic loadings of 50 to 75 I.gpcd for communities of similar size.

The total organic fraction consisting of suspended and dissolved organic solids was found to average 0.185 lbs. pcd. Again this compares to average North American design figures of 0.2 lbs. pcd normally used if no data is available. (ASCE, 1959)

#### 4.1 Biochemical Oxygen Demand

FIGURE 19 indicates typical diurnal BOD-5 variations of both plant influent and effluent. Influent BOD-5 concentrations varied from a maximum of 465 mg/l to a minimum of 100 mg/l averaging 297 mg/l. By chance the average influent BOD-5 values of FIGURE 19 and the three composite samples shown in TABLE III are in exact agreement. Effluent BOD-5 values range from a maximum of 111 mg/l to a minimum of 62 mg/l and average 73 mg/l. During Phase II of the testing program the treatment efficiency approximates only 75% which is poor for any extended aeration complete mixing activated sludge process. The major contributing factor to this poor treatment efficiency experienced can be attributed to the shut-down of the treatment plant in early December with start-up not occurring until December 20th. As shown in FIGURE 19 the plant effluent did not stabilize





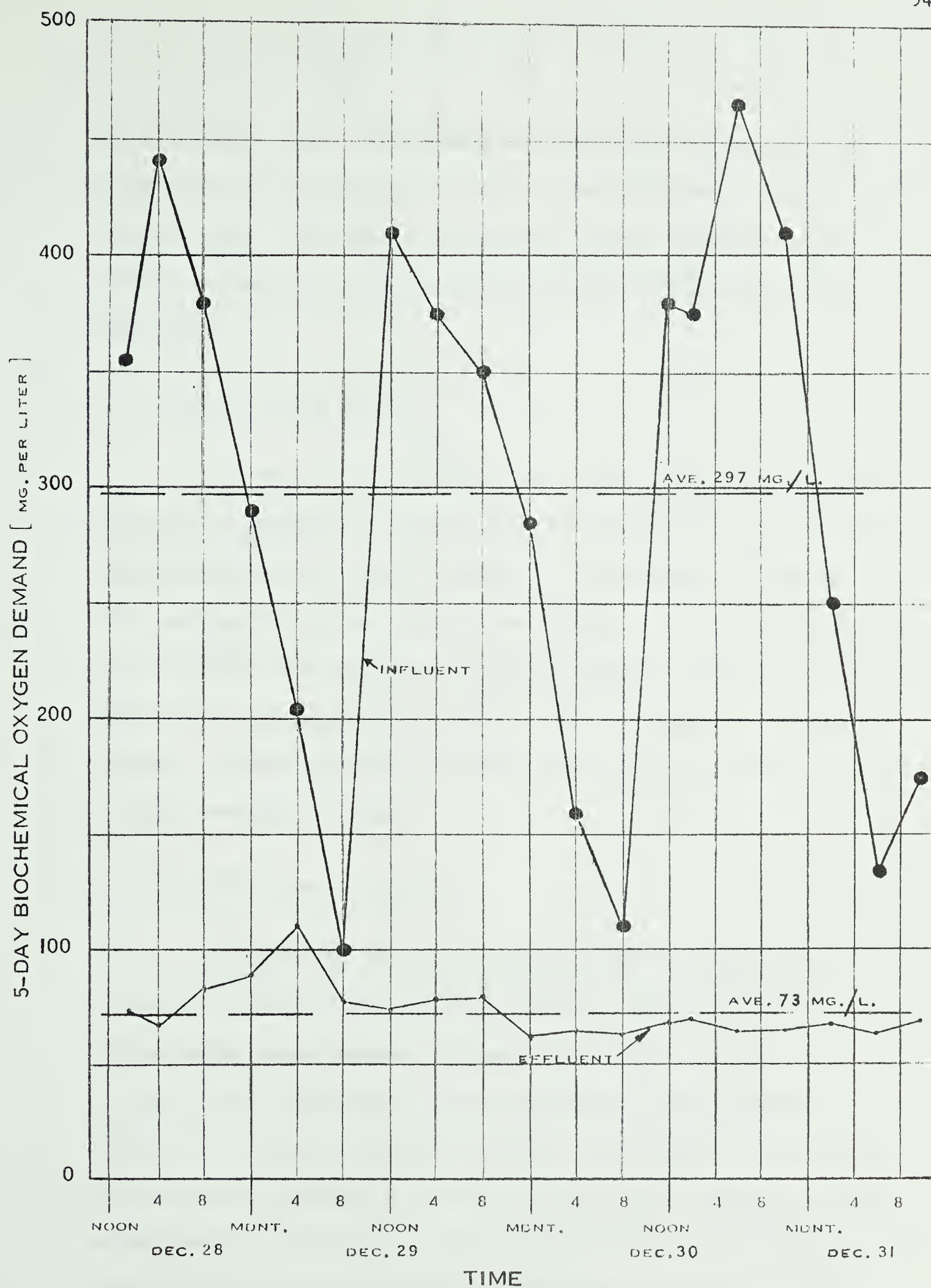


FIGURE :19 DIURNAL 5-DAY BIOCHEMICAL OXYGEN DEMAND VARIATIONS



until December 30th. Subtracting a 2-day detention time for the system the mixed liquor would appear to be stabilized on about December 28th. This can be interpreted that at least a one week period is required for the oxidation ditch to attain equilibrium conditions.

#### 4.2 Chemical Oxygen Demand

Diurnal COD variations of both plant influent and effluent are shown in FIGURE 20. The influent COD concentrations vary from a high of 1050 mg/l to a low of 96 mg/l. The average COD for the 4-day testing period was found to be 542 mg/l. This represents only 77% of the COD value determined from the composite sample analysis (TABLE III). No explanation for this wide divergence of values is offered. The effluent COD concentrations varied from 269 mg/l to 150 mg/l averaging 223 mg/l.

#### 4.3 Volatile Suspended Solids

Analyses for volatile suspended solids (VSS) of both influent and effluent are shown in FIGURE 21. The influent VSS concentration varied between 390 mg/l and 32 mg/l averaging 203 mg/l, while the effluent VSS concentration had a range of 130 mg/l to 24 mg/l, averaging 71 mg/l. The influent and effluent VSS determined represent 85% and 74% of the total suspended solids respectively. A declining trend of effluent VSS is noticeable; however it is felt that the testing period was too short to draw



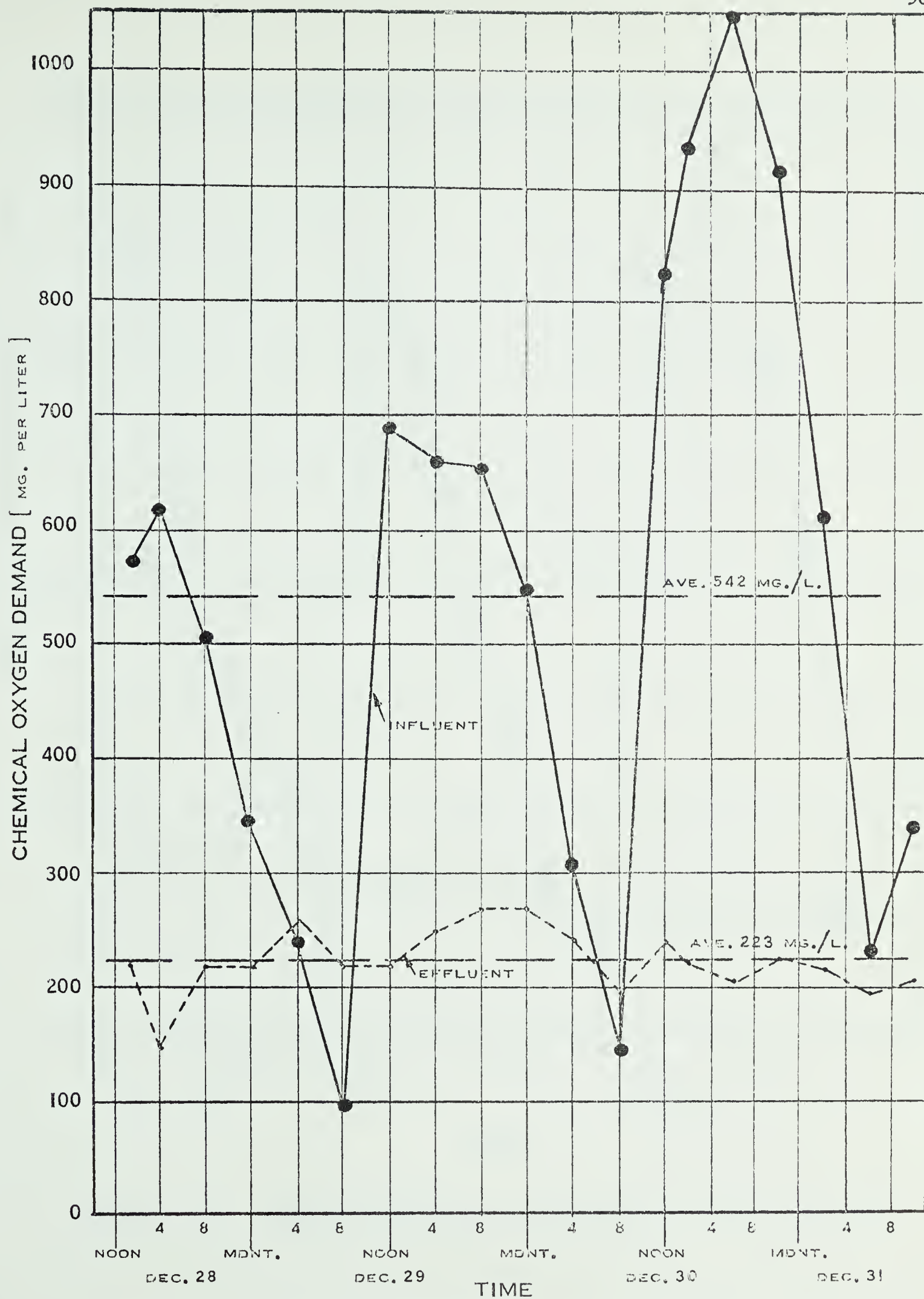


FIGURE : 20 DIURNAL CHEMICAL OXYGEN DEMAND VARIATIONS





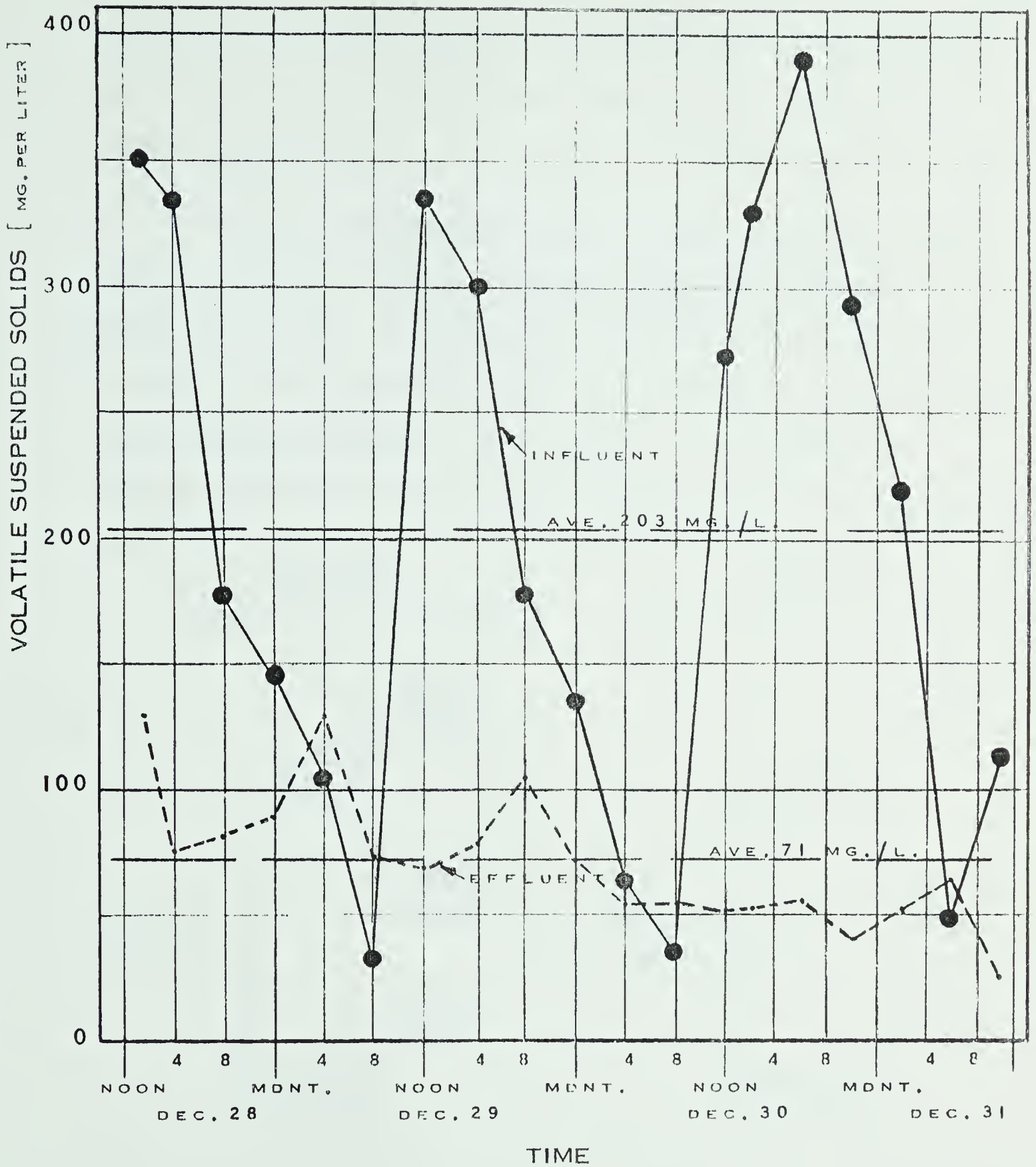


FIGURE : 21 DIURNAL VOLATILE SUSPENDED SOLIDS VARIATIONS



any definite conclusions regarding possible trends. As in FIGURE 19 the stabilizing of the effluent VSS values can be noticed to begin on about December 30th.

FIGURE 22 indicates that even if a high reduction in BOD-5 and SS is achieved, a high percentage of the organic substance remains in solution. Primarily this is indicative of a transformation process where the organic matter has been converted into non-biochemical oxygen consuming substances. The main part of the organic and inorganic substances remains in the effluent although in a transformed state.

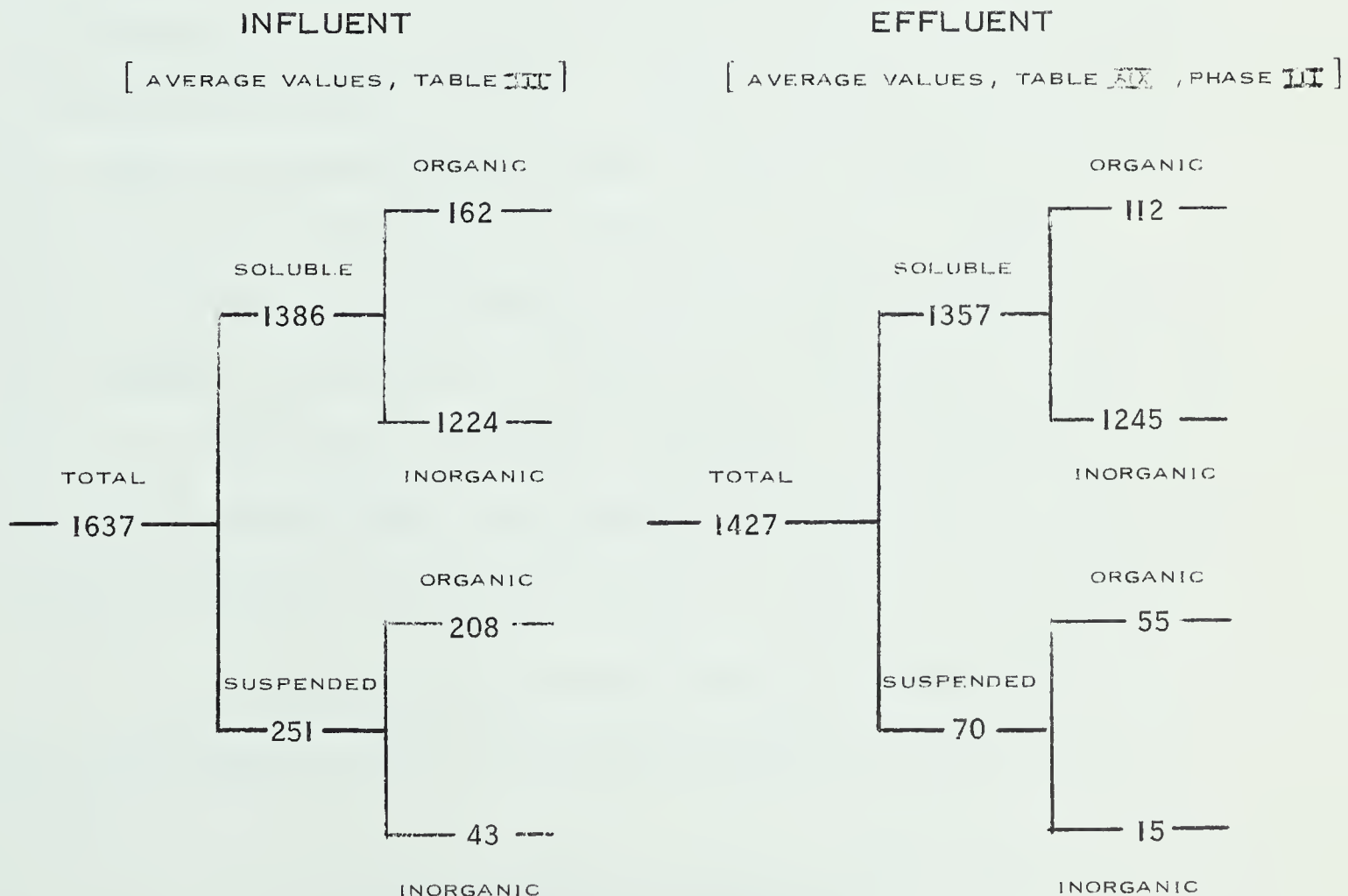


FIGURE 22 SOLIDS TRANSFORMATION PATTERN



#### 4.4 Phosphates

Influent diurnal phosphate variations are shown in FIGURE 23 range from 24 mg/l to 0.8 mg/l and average 5.8 mg/l. Again the uniformity of the effluent can be noted after December 29th, when the effluent phosphate concentration drops below the 4.6 mg/l average value for the testing period and remains constant.

#### 4.5 Nitrogenous Compounds

Effluent ammonia, nitrite and nitrate analyses carried out indicate that only a small portion of the influent ammonia concentration was converted to nitrite and nitrate. FIGURE 24 shows an average influent ammonia concentration of 34.8 mg/l and the effluent ammonia concentration was only reduced to an average of 29.3 mg/l. Effluent nitrite and nitrate concentrations are shown in FIGURES 41 and 42 in the APPENDIX. Since nitrite is rapidly and easily converted to nitrate it is only found in very small quantities. Its presence in concentrations greater than a few thousands of a milligram per liter is generally indicative of active biological processes in the substrate.

It is known that nitrification depends chiefly on three factors which would provide a favorable environment for nitrifying bacteria:

1. the oxygenation capacity to load ratio
2. the mixed liquor temperature
3. the MLSS content of the system





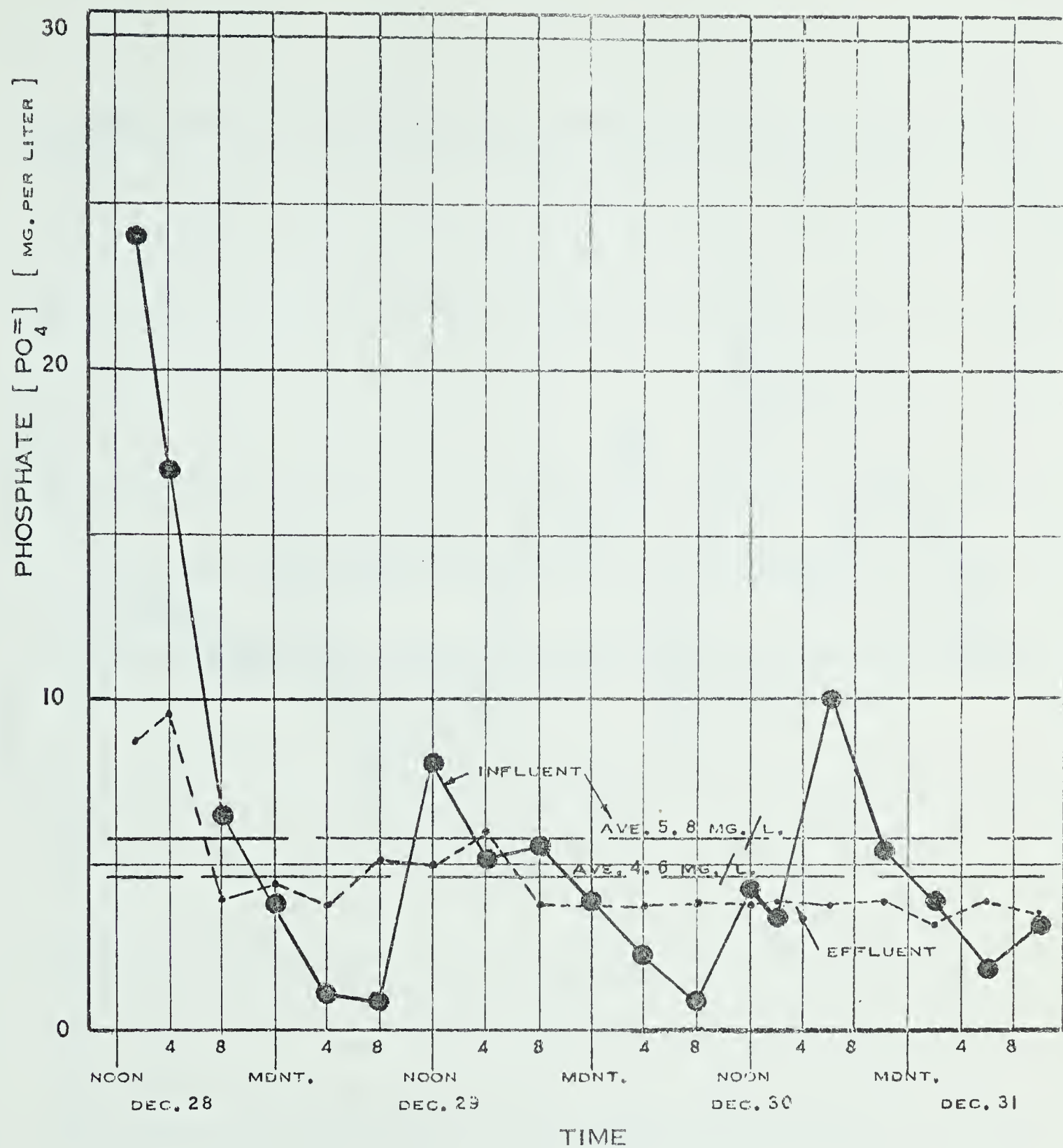


FIGURE : 23 DIURNAL PHOSPHATE [  $\text{PO}_4$  ] VARIATIONS



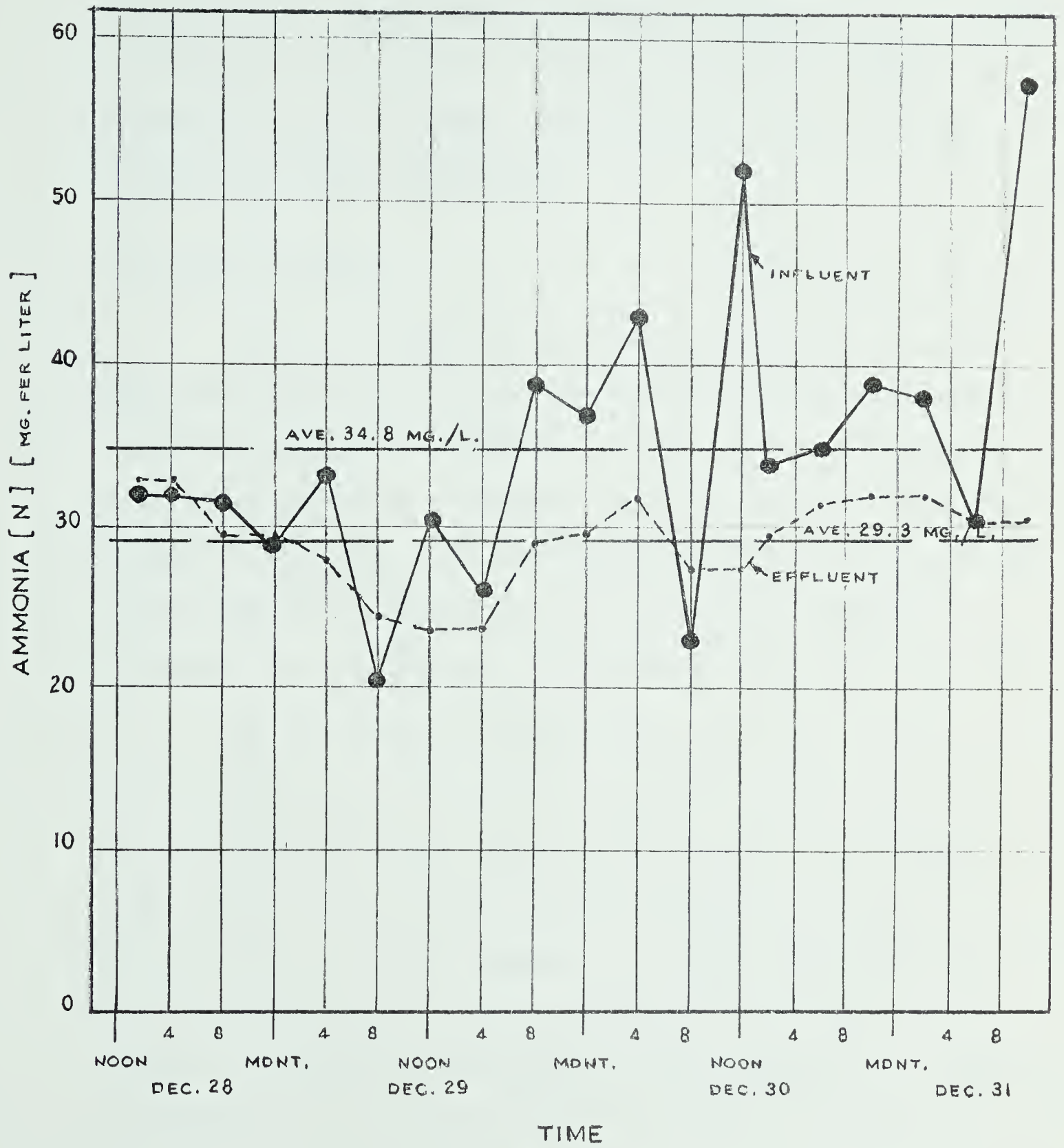


FIGURE : 24 DIURNAL AMMONIA [N] VARIATIONS



Whereas it is known that the effect of temperature on the active microbial mass is slight (McKinney 1962) nitrification will be retarded at temperatures other than the optimum nitrification temperature of 55 degrees Fahrenheit. A MLSS content exceeding 3000 mg/l is also favorable to complete nitrification.

#### 4.6 Dissolved Oxygen

Dissolved oxygen determinations at various points in the ditch were carried out at 4-hour intervals at 8 ditch locations and are shown as profiles in FIGURE 25. Since the rotors are on time-clocks operating 24 hrs. per day the oxygenation capacity would remain constant if the liquid ditch level would remain constant as well. As already noted it was observed that the liquid depth fluctuated throughout the day through an average range of 0.66 feet.

The mixed liquor dissolved oxygen (DO) content is a maximum at approximately 8:30 a.m.. It appears that the DO will not increase much beyond 6 mg/l since at that time the average oxygen uptake at each rotor is only 0.45 mg/l as compared to 0.80 mg/l at the other times. This also compares to the oxygen absorption study with Kessener brushes as described in CHAPTER II and FIGURE 4. The response of decreasing DO throughout the ditch from 8:30 a.m. onward is expected, as the incoming organic load exerts an increased oxygen demand on the system.

The average daily DO increase through rotors 7 and 8 was





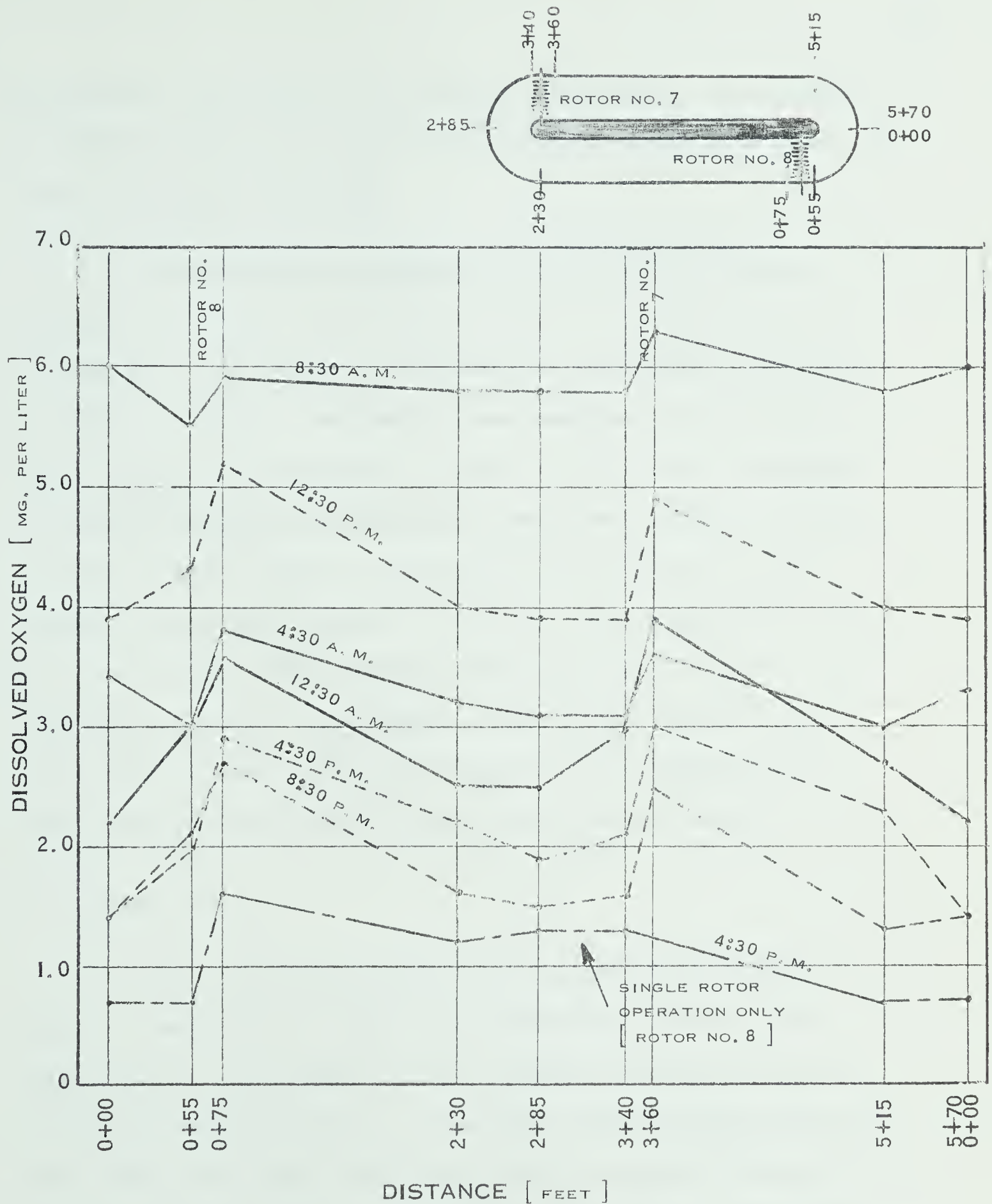


FIGURE : 25 MIXED LIQUOR DISSOLVED OXYGEN PROFILES



determined to be 0.78 mg/l and 0.70 mg/l respectively. As indicated in FIGURE 25 single rotor operation is barely adequate to maintain any dissolved oxygen in the ditch.

FIGURE 26 shows the diurnal DO variations indicating a minimum DO of 1.8 mg/l which occurs at 8:30 p.m. and the subsequent DO recovery of the system to an approximate maximum DO concentration of 6 mg/l by 8:30 a.m. Average DO concentrations in the treatment plant effluent are also shown. It should be noted that the effluent DO measured is really an indication of aeration conditions occurring 53 hours earlier. (44.6 hrs. detention in the ditch and 8.5 hrs. detention in the settling tank). However, it was assumed that the DO concentrations in both effluent and mixed liquor would not alter their diurnal pattern significantly during a short period of time since daily influent organic and hydraulic plant loadings did not change significantly and rotor running time remained constant.

#### 4.7 Temperatures

One of the most important factors affecting microbial growth is temperature. It has been observed that bacteria grow quite slowly at low temperatures but increase their rate of reaction as the temperature increases. It has been generally stated that the rate of microbial growth doubles with every 10 degrees Centigrade increase in temperature up to the limiting temperature. The growth reactions are similar to normal chemical reactions which follow definite patterns. Two patterns which are interposed with micro -organisms are the



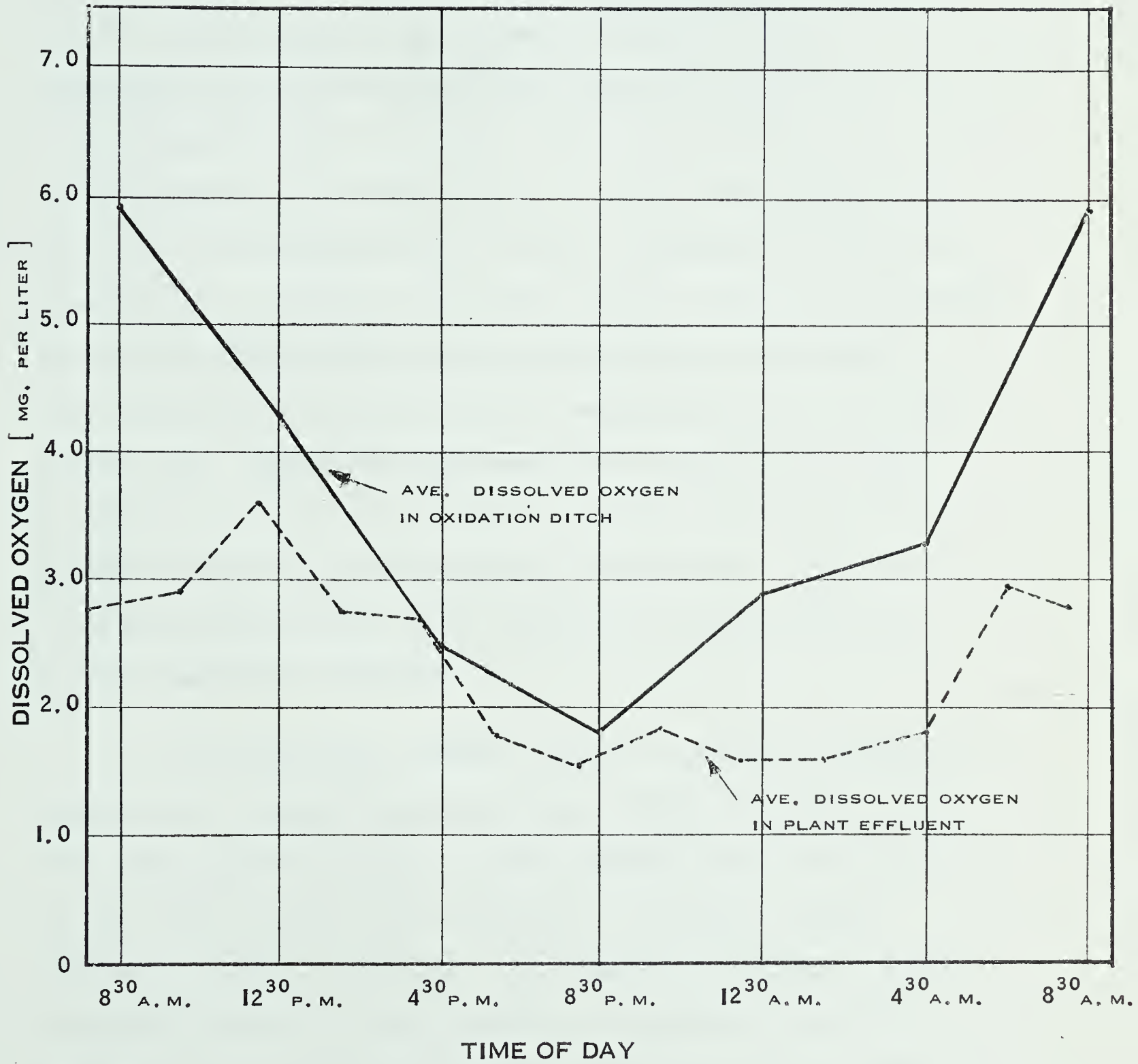


FIGURE : 26 DIURNAL DISSOLVED OXYGEN VARIATIONS



increased rate of reaction with increased temperature and denaturation of specific proteins at definite temperatures. When these two phenomena are overlaid, at low temperatures the denaturation reaction is insignificant. As the temperature approaches +35 degrees Centigrade the denaturation reaction soon predominates and the micro-organisms' rate of growth rapidly falls off to zero. Most micro-organisms cannot grow at low temperatures since the water which makes up 80 percent of the cell freezes and prevents further reaction. Exceptions are some micro-organisms with a minimum of water which have the ability of withstanding temperatures slightly below freezing and are known as psychophilic micro-organisms. The metabolic rate of psychophilic micro-organisms is very slow. Collectively low temperatures result in a slow rate of metabolism but the rate of metabolism can be raised to a certain degree by increasing the micro-organism concentration. During Phase II of the testing program various temperature readings were taken and are summarized in TABLE IV.

The temperature readings indicate a temperature decrease of approximately 7 degrees Fahrenheit between influent and mixed liquor. This small decrease even after two days detention time in the ditch is due to the relatively small temperature differential between influent and ambient temperature. The temperature difference between the mixed liquor and the plant effluent is negligible, and can be attributed to the reduced heat loss due to a covered settling tank.





TABLE IV

## TEMPERATURE SUMMARY - PHASE II

Date	Mean Temperatures (°F)			
	Air	Influent	Mixed Liquor	Effluent
Dec. 28	30	43	35.5	35.5
Dec. 29	26	44	37.5	36.5
Dec. 30	28	43	35.5	35.5
Dec. 31	24	44	35.5	35.5

The influence of temperature on bio-oxidation processes has been formulated by many researchers but observations indicate that the temperature effect is less than that estimated from the various available equations. McKinney (1962) also notes that the influence of temperature on a complete-mixing extended aeration system is small. Whether these claims hold true for temperatures slightly above freezing bears further investigation.

#### 4.8 Settleable Solids

The mixed liquor settleable solids for the four day period averaged 70 ml. With a MLSS concentration of 2260 mg/l the SVI was calculated to be 31. This indicates that as during Phase I of the testing program the sludge recirculation is adequate to ensure a high MLSS build-up in the ditch.



#### 4.9 Phase II - Summary

The various treatment parameters observed during Phase II of the testing program are summarized in TABLE V.

TABLE V

##### 4-Day Continuous Testing Program Summary

##### Phase II

Test	Influent mg/l	Effluent mg/l	Mixed Liquor mg/l
BOD-5	297	73	-
COD	542	223	-
TSS	239	96	2260
VSS	203	71	-
$\text{PO}_4^{=}$	5.8	4.6	-
$\text{NH}_3\text{-N}$	34.8	29.3	-
DO	-	2.3	3.45
Settleable Solids (ml)	-	-	70
SVI	-	-	31

#### 5. PHASE III - Extended Testing Program

The extended testing program continued from January 3rd, 1967 to February 20th, 1967 and consisted of analyses of the plant effluent as indicated in TABLE I. During this period daily samples were taken at the settling tank effluent weir and stored outdoors



until six samples had been collected at which time they were shipped by bus to Edmonton. The samples arrived in a frozen state and had to be thawed before analysis. No apparent effect of prolonged storage at below freezing temperatures on the BOD and ammonia analyses was evident.

### 5.1 Biochemical Oxygen Demand

One of the objectives of this thesis was to ascertain if the treatment plant efficiency as measured by the effluent BOD-5 and VSS are influenced by low temperatures. FIGURE 27 shows that during this phase of the testing program temperatures varied from +32 degrees F. to -21 degrees F. and averaged +12.5 degrees F.. The BOD-5 effluent analyses average 46 mg/l and do not indicate any influence of temperature on BOD-5. Hence it is concluded that a complete mixing extended aeration activated sludge system is not temperature sensitive. McKinney (1962) illustrates mathematically that little variation in the oxygen demand of complete mixing activated sludge systems with changes in temperatures will occur.

### 5.2 Chemical Oxygen Demand

The COD analyses as noted in FIGURE 28 indicate a range of values from 362 mg/l to 138 mg/l with an average value of 210 mg/l. The average COD value compares with the average value of 216 mg/l obtained in Phase I even though the average BOD-5 values differ by 50%.





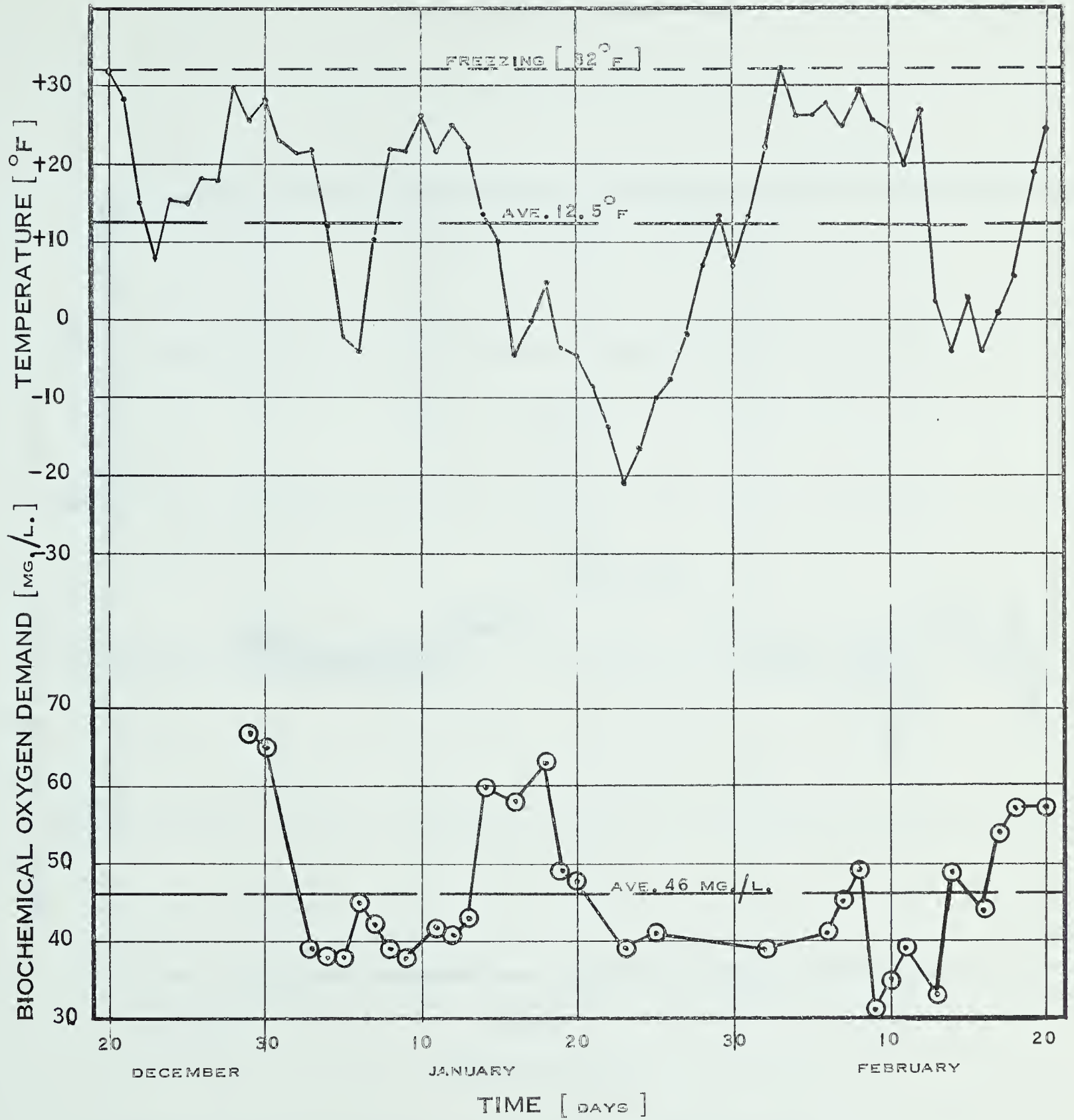


FIGURE : 27 MEAN DAILY AMBIENT TEMPERATURE AND EFFLUENT 5-DAY BIOCHEMICAL OXYGEN DEMAND



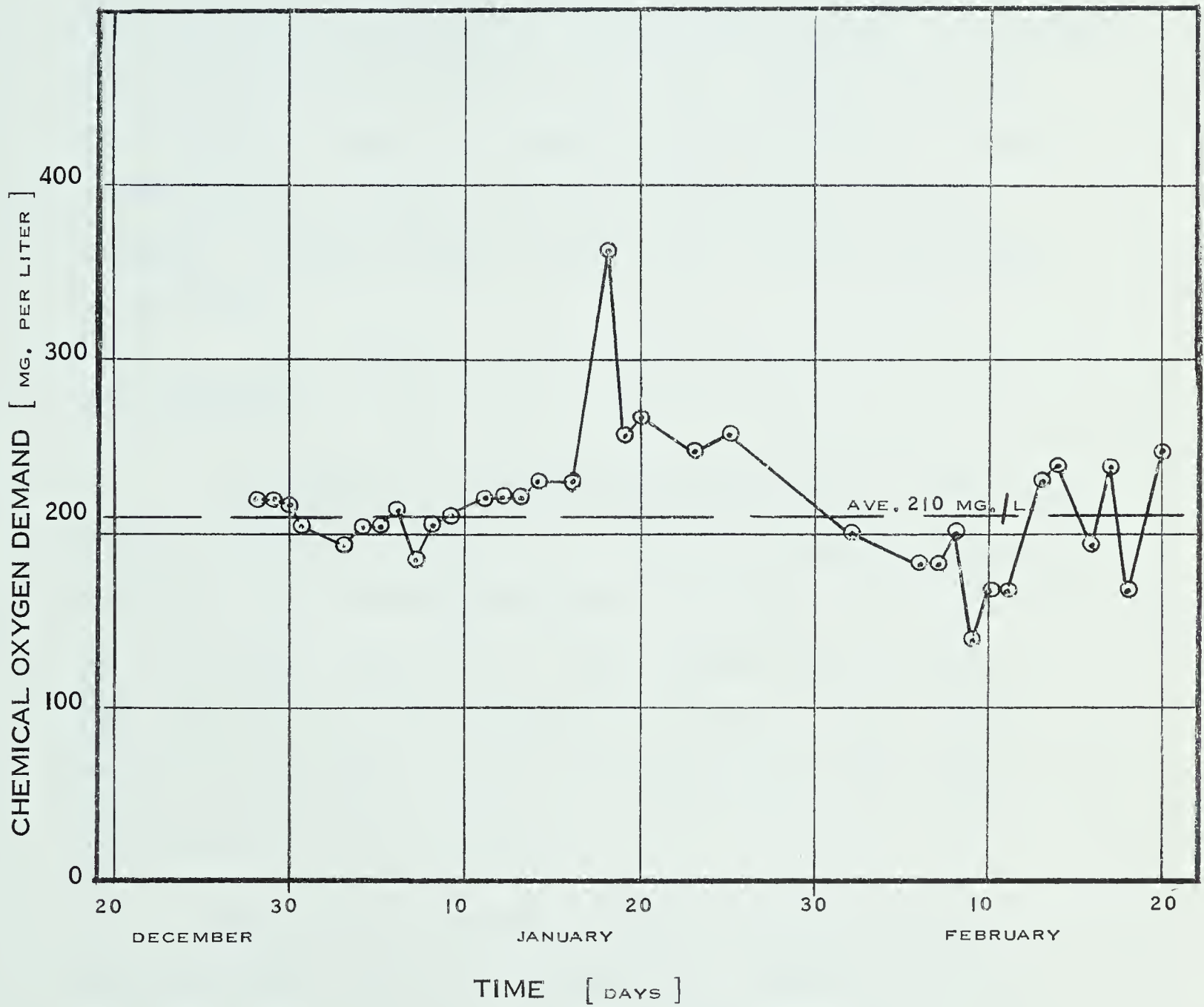


FIGURE : 28 EFFLUENT CHEMICAL OXYGEN DEMAND



### 5.3 Volatile Suspended Solids

FIGURE 29 indicates the extremely large fluctuations in effluent volatile suspended solids. Again no correlation between VSS fluctuations and temperature fluctuations is evident. The average VSS concentration was determined as 55 mg/l. Fluctuations ranged from 120 mg/l to 4 mg/l. It appears though that a definite trend of decreasing VSS is present. As will be demonstrated in a later experiment increased removal of VSS is due to the build-up of MLSS in the ditch.

### 5.4 Phosphates

The results of effluent phosphate determinations made are shown in FIGURE 30. Due to lack of time in performing all analyses immediately upon receiving the sample, most samples were stored near zero degrees Centigrade for later phosphate determinations. The implications of this procedure did not become apparent until samples not previously analysed for phosphates were taken from storage and subjected to analysis.

As indicated in FIGURE 30 samples taken and analysed immediately upon receipt had an average  $\text{PO}_4^{=}$  concentration of 3.4 mg/l while samples stored for a period of 3 weeks showed an average of 20.8 mg/l  $\text{PO}_4^{=}$ . The remaining samples having been stored from 5.5 to 12 weeks had stabilized at an average  $\text{PO}_4^{=}$  concentration of 44 mg/l. In all instances the orthophosphate test as outlined in



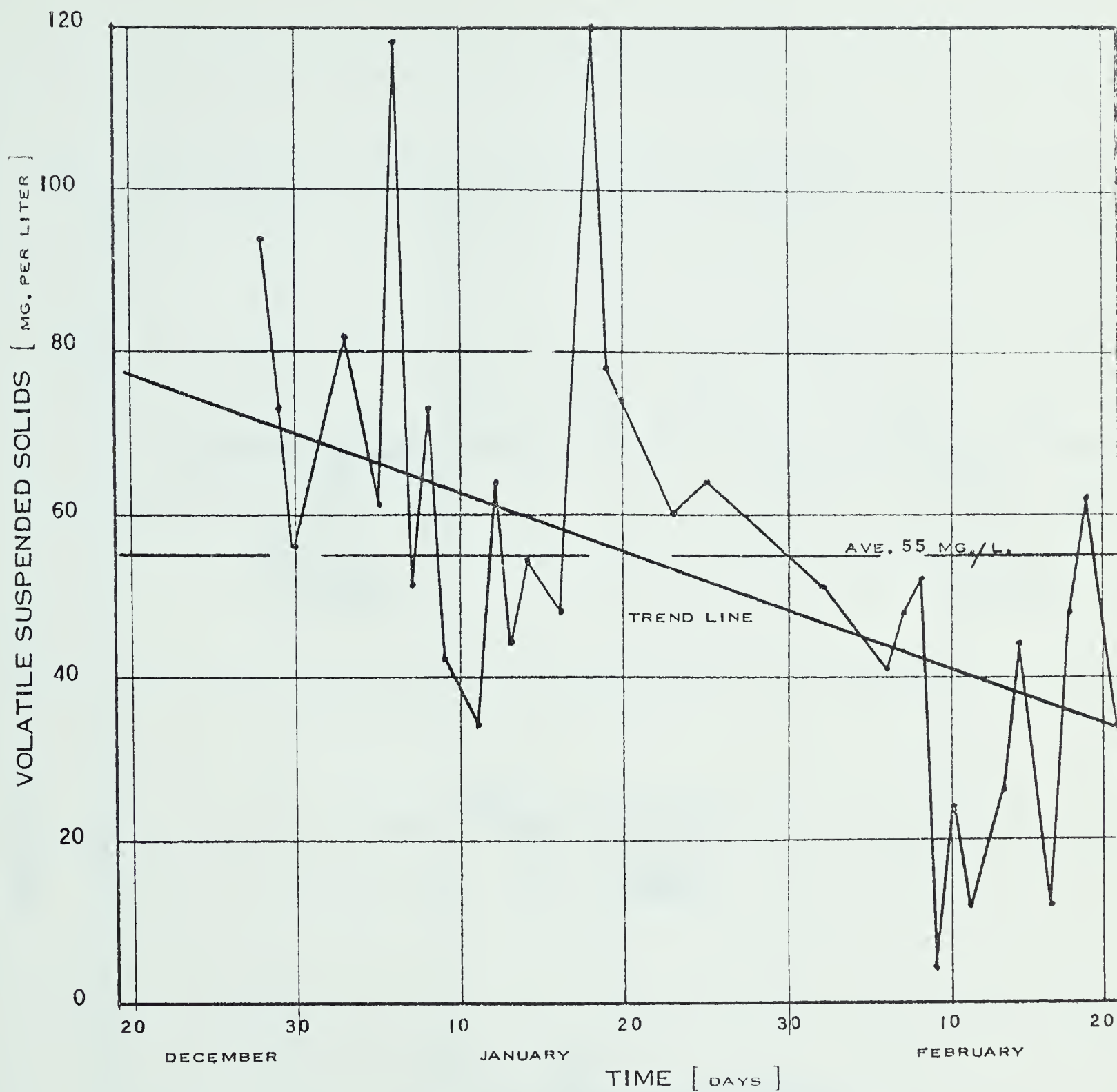


FIGURE : 29 EFFLUENT VOLATILE SUSPENDED SOLIDS





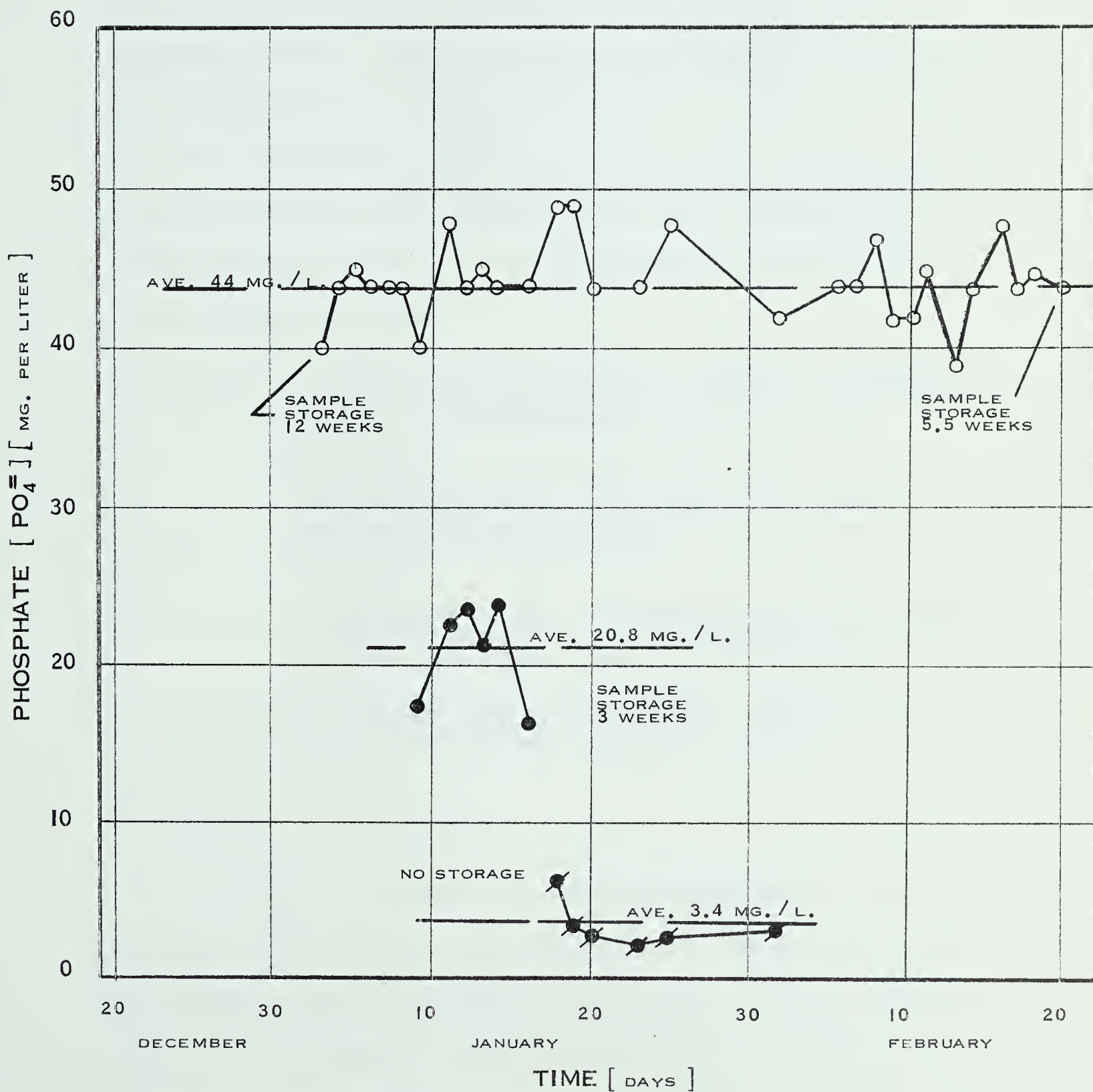


FIGURE : 30 EFFLUENT PHOSPHATE [  $\text{PO}_4^=$  ]



the 12th Edition of Standard Methods was used.

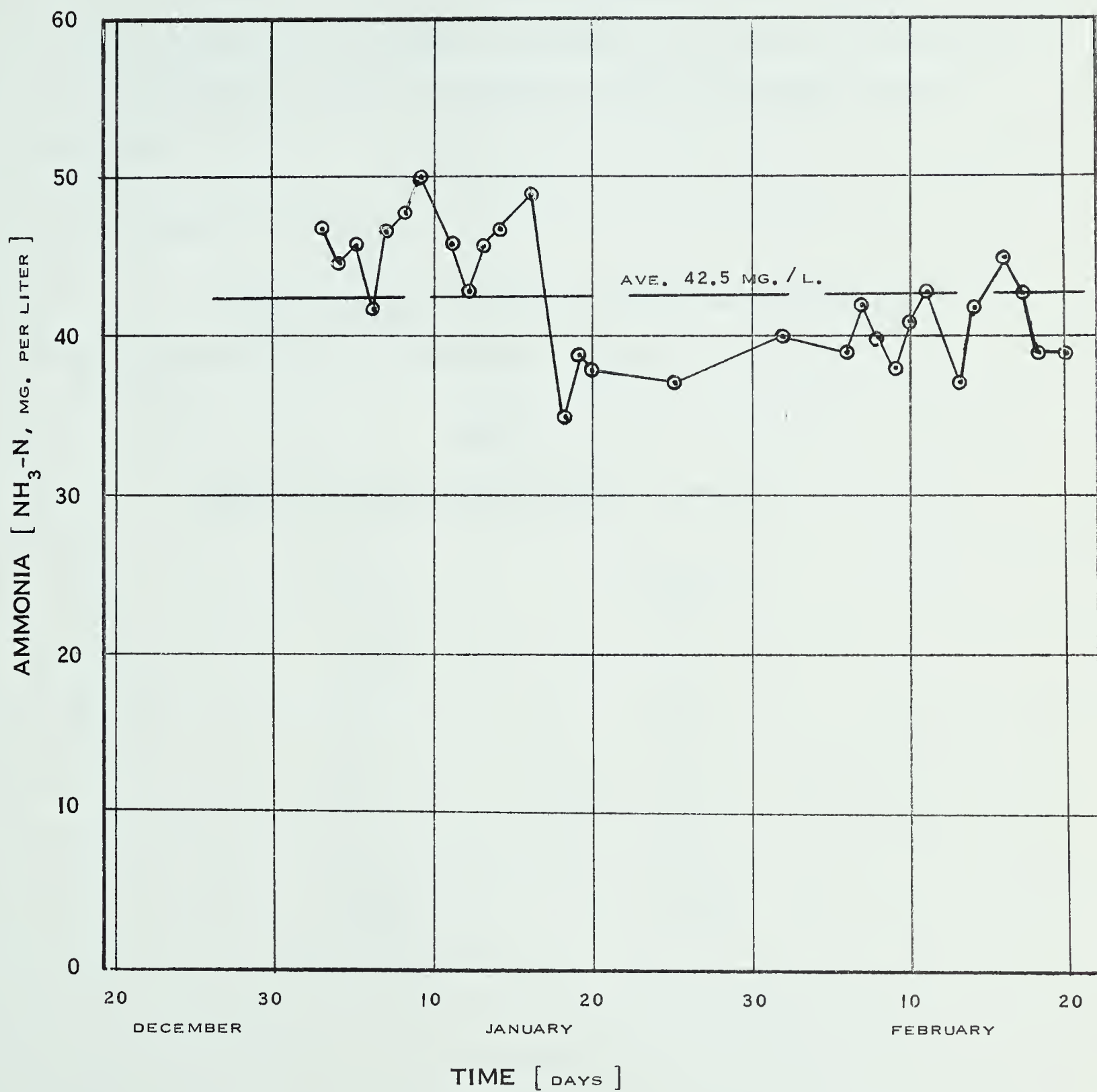
The large differences in results can be explained by the gradual hydrolysis of all phosphates (molecularly dehydrated phosphates) in aqueous solutions and reversion to the ortho form from which they were derived. The rate of hydrolysis is beneficially influenced by high temperatures, low pH or the presence of bacterial enzymes. Since the temperature was low, the pH near neutral only the action of enzymes appears to be responsible for the polyphosphate to orthophosphate conversion. From the phosphate determinations four conclusions may be drawn:

1. In order to measure the total inorganic phosphates in a waste sample, procedures to measure polyphosphates instead of orthophosphates must be employed.
2. To determine orthophosphates the sample should be analysed immediately upon receipt. Storage near zero degrees Centigrade does not prevent the hydrolysis of polyphosphates.
3. It appears that after a minimum period of 5.5 weeks at temperatures near zero degrees Centigrade all polyphosphates have hydrolysed to orthophosphates.
4. The average total orthophosphate concentration for the oxidation ditch effluent was 44 mg/l.

## 5.5 Ammonia

FIGURE 31 indicates ammonia concentrations for the treatment plant effluent, which vary between 35 mg/l and 50 mg/l. The average value was determined as 42.5 mg/l. The data indicates that the average amount of ammonia was lower for the period



FIGURE : 31 EFFLUENT AMMONIA [  $\text{NH}_3\text{-N}$  ]





starting January 20th than for the earlier portion of the Phase III test period. This would lead one to suspect that increased nitrification of the effluent is occurring, (i.e. nitrate concentrations are increasing). This is a desirable quality in any waste treatment process.

#### 5.6 Phase III - Summary

Various treatment parameters observed during Phase III of the testing program are summarized in TABLE VI

TABLE VI

Extended Testing Program Summary - Phase III

Test	Effluent mg/l
BOD-5	46
COD	216
TSS	70
VSS	55
PO <sub>4</sub>	44*
NH <sub>3</sub> -N	42.5

\* See Section 5.4



## 6. Theoretical and Measured Effluent Characteristics

The measured effluent characteristics and theoretically derived values using the mathematical procedures for analysis as presented in CHAPTER III are summarized as follows:

TABLE VII

### THEORETICAL AND ACTUAL EFFLUENT CHARACTERISTICS

Phase	MLVSS mg/l	Ma mg/l	x	Effluent BOD-5		Effluent VSS	
				Theoretical mg/l	Actual mg/l	Theoretical mg/l	Actual mg/l
I	2508*	562	0.049	16.9	32	123	49
II	1540	508	0.088	17.8	73	135	71
III	2195**	550	0.057	19.2	46	125	55

\* Nov. 10/66

\*\* estimated

(Mi)i = 50 mg/l (assumed)

The analyses demonstrate that MLVSS is not a good measure of the active microbial mass in the oxidation ditch. Even though the MLVSS increase, the percent active solids decrease from 33% in Phase II to 22% in Phase I.

Theoretical BOD-5 values for Phases I and III average 48% of the actual values. Phase II BOD-5 data was disregarded since the treatment plant had only been in operation 1 week prior to Phase II and had not sufficient time to build up a high MLSS concentration.



It is postulated that the large difference between the actual and theoretical BOD-5 values is due to any one or a combination of the following factors:

1. the high effluent BOD was due to a high F/M ratio. (i.e. not enough solids were built up in the ditch for a satisfactory metabolic balance)
2. some of the constants in McKinney's mathematical model do not hold for low temperature mixed liquor solutions.
3. the assumed value for the inert volatile organics  $(M_i)_i$  is too low.

The average measured effluent VSS concentration for all 3 phases is only 45% of the theoretical value. Suspended solids removal is a physical operation not involving dependance on biological activity. The effect of higher VSS removal at higher MLVSS is illustrated by comparing measured effluent VSS for Phases I and III to Phase II. The difference between actual and theoretical effluent VSS concentrations may be attributed to the better settling characteristics and settling tank efficiency than was theoretically anticipated.



## CHAPTER VI

### SPECIAL STUDIES

Special studies carried out in connection with this thesis consisted of effluent coliform determinations, mixed liquor settling studies, oxidation reduction potential survey, oxidation ditch velocities and oxygen transfer coefficient.

#### 1. Effluent Coliform Concentrations.

Spot checks on coliform concentrations in the effluent were made using a Millipore Field Monitoring Kit. Serial dilutions of 1 ml effluent samples were filtered and incubated for 18 hours at 35 degrees Centigrade on M-Endo broth medium. All organisms producing a dark, generally purplish-green colony with a metallic sheen covering either the entire colony or only a part were counted and expressed as the estimated coliform density per ml in accordance with the following relationship:

$$\text{estimated coliform colonies/ml} = \frac{\text{coliform colonies counted}}{\text{dilution} \times \text{ml sample filtered}}$$

As pointed out in Standard Methods 12th Edition, the coliforms counted in the group as previously defined is not necessarily the same as that described in the multiple-tube fermentation technique. Its sanitary significance however, is believed to be the same.





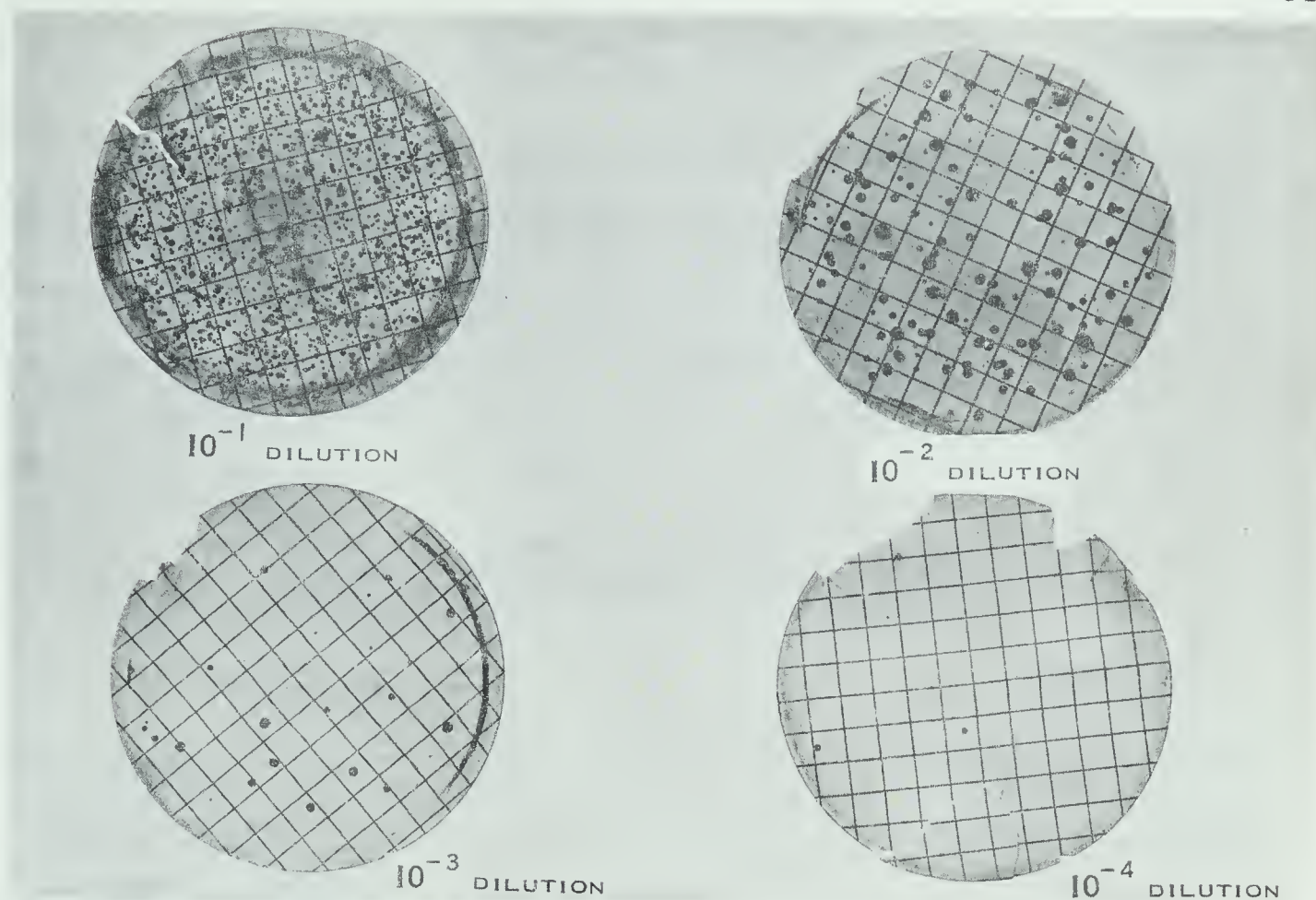


FIGURE : 32 COLIFORM COLONY PRODUCTION  
[ EFFLUENT SAMPLE, FEB. 2, 1967 ]

An example of colony production at various dilutions is shown in FIGURE 32.

TABLE VIII

EFFLUENT COLIFORM CONCENTRATIONS

Date	Effluent Temperature (°F)	MPN. organisms/ml
Feb. 2	35	2100
Feb. 10	36	2000
Feb. 16	36	2000



The average effluent coliform concentration as indicated in TABLE VIII appears to fall into the same range as for conventional waste stabilization pond effluent. The range of coliform organism concentrations in raw sewage and effluents from various types of sewage treatment facilities is illustrated in TABLE IX.

TABLE IX

## COLIFORM CONCENTRATIONS (Higgins, 1966)

Description	MPN Org/ml
Raw sewage	250,000 - 1,000,000
Primary Effluent	100,000 - 500,000
<u>Secondary Effluent:</u>	
High Rate Filters	50,000 - 250,000
Trickling Filters or Activated Sludge	3,000 - 10,000
<u>Septic Tanks:</u>	
24 hr. retention	10,000 - 500,000
<u>Anaerobic Lagoons:</u>	
3 month retention	5,000 - 250,000
Conventional Waste Stabilization Ponds	50 - 5,000
High Rate oxidation pond (Oswald type)	0 - 1,000

In instances where the effluent coliform count constitutes a potential health hazard, effluent chlorination must be practised.



## 2. Activated Sludge Sedimentation Studies

The suspended solids analyses carried out in Phases I to III of the Testing program indicated that even though the MLSS concentration in the ditch was still relatively low, (less than 3000 mg/l) the effluent suspended solids concentration was comparatively high (70 - 96 mg/l). Some concern was expressed as to what the effluent suspended solids concentration might be once the oxidation ditch is operating in the 4000 - 6000 mg/l suspended solids range.

It has been observed that the clarification of dilute suspensions of flocculating particles is not only a function of the settling properties of the particles but also of the flocculating characteristics of the suspension. No satisfactory method for evaluating the flocculation effect on sedimentation is available and it is therefore necessary to perform settling-column analyses in order to measure the factors affecting the sedimentation of flocculent suspensions. Fitch (1957) summarizing the work done by Hazen, Coe, Clevenger and Camp in sedimentation process fundamentals states that particles will settle out of a suspension in one of four different ways, depending upon the concentration of the suspension and the flocculating properties of the particles. The manner in which particles having little tendency to flocculate settle out of a dilute suspension is identified as class-1 clarification. If the particles flocculate, the sedimentation regime in a dilute suspension is labeled class-2 clarification. In flocculent suspensions of intermediate concentration,





particles are close enough to permit interparticle forces to hold the particles in a fixed position relative to each other. As a result, the mass of particles subsides as a whole in a regime described as zone settling. Compression occurs when the concentration becomes high enough so that the particles come into actual contact with each other, and the weight of the particles is supported, in part, by the structure formed by the compacting mass.

This study deals with class-2 clarification and zone settling. Class-2 clarification applies to dilute suspensions of flocculating particles. Rich (1961) notes that particles subsiding from such suspensions overtake and coalesce with smaller particles to form particles which settle at rates higher than did the parent particles. The greater the liquid depth, the greater will be the opportunity for contact, hence the suspended solids removal is dependent both on the clarification rate and on depth.

In the zone settling regime, the particles are locked into a floc structure and the mass of particles settles as a blanket forming a distinct interface between the floc and the supernatant. All particles are constrained to settle at the same rate which is a function of the initial solids concentration. The concentration of solids during zone settling will remain constant until the settling interface approaches an interface of critical concentration. As the depth of settled sludge solids increase, the settling floc begins to press on layers below and a transition zone occurs. Through



the transition zone the settling velocity will decrease due to the increasing density and viscosity of the suspension surrounding the particles. A compression zone occurs when the floc concentration becomes so great as to be mechanically supported by the layers of the floc below. The solids concentration in the compression zone will be related to the depth of sludge and the detention of the solids in this zone.

The laboratory study consisted of measuring the concentration of suspended solids at various depths and time intervals in an acrylic settling column as shown in FIGURE 33. The column was 8'-6" in height, 150 mm O.D. and 10 mm wall thickness. One-inch diameter holes for the installation of sample taps were drilled at 2'-0" intervals. Each tap consisted of 8 mm I.D. glass tubing which extended to the centre of the settling column. A thermometer was placed in a separate tap 3'-0" below the zero level of the settling column. A combination drain and air supply tap was also provided at the bottom of the column. The function of the air supply was to keep the mixed liquor in suspension after filling the column and before starting the test. Samples were drawn off into 125 ml plastic bottles at various time intervals. Immediately before taking each sample the liquid in each tap was wasted. The sludge used in this study was obtained from the Olds pollution control centre and gently aerated with compressed air while being transported to Edmonton in 18 liter plastic carboys. Approximately 4 hours elapsed between taking the



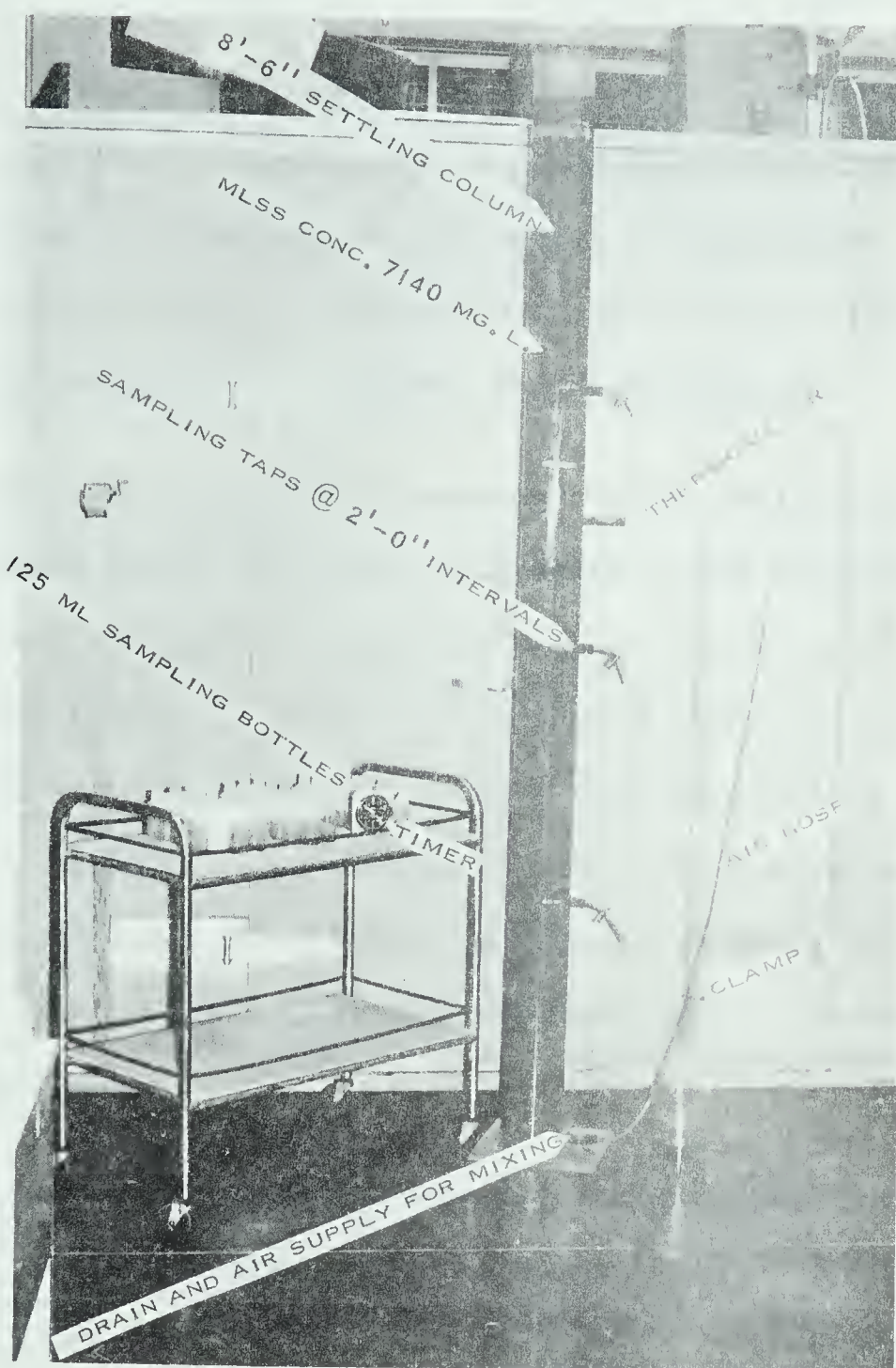


FIGURE : 33      LABORATORY SETTLING COLUMN





activated sludge sample and running the first experiment. A DO test showed that the sludge mixture was still aerobic ( $\approx 0.2$  mg/l DO).

Two test runs were made; one at a MLSS concentration of 4530 mg/l, the other at 7140 mg/l MLSS. The high concentration of MLSS was obtained by decanting the top 6 feet of supernatant after completing the first test run and refilling the settling column with new activated sludge of 4530 mg/l MLSS concentration.

FIGURE 34 indicates 90% suspended solids removal paths for the activated sludge mixtures at suspended solids concentrations of 4530 mg/l and 7140 mg/l. The solids removal path for the 4530 mg/l MLSS concentration is typical of flocculent settling in that it is curvilinear in appearance. It shows that the settling velocity is constant to approximately 70 minutes. After 70 minutes detention time flocculation occurs increasing the settling velocity of the particles as they settle through the tank depth due to coalescence with other particles.

In the second settling experiment using a MLSS concentration of 7140 mg/l, flocculent settling characterized by the curvilinear settlings path persists for the first 30 minutes. Thereafter hindered and zone settling appears to occur for the next 40 minutes as indicated by the low settling velocity. The last part of the curve appears to be indicative of a compression zone as the floc concentration has become too great to be mechanically supported by the layers of the floc below. Settling paths for the VSS were also determined and





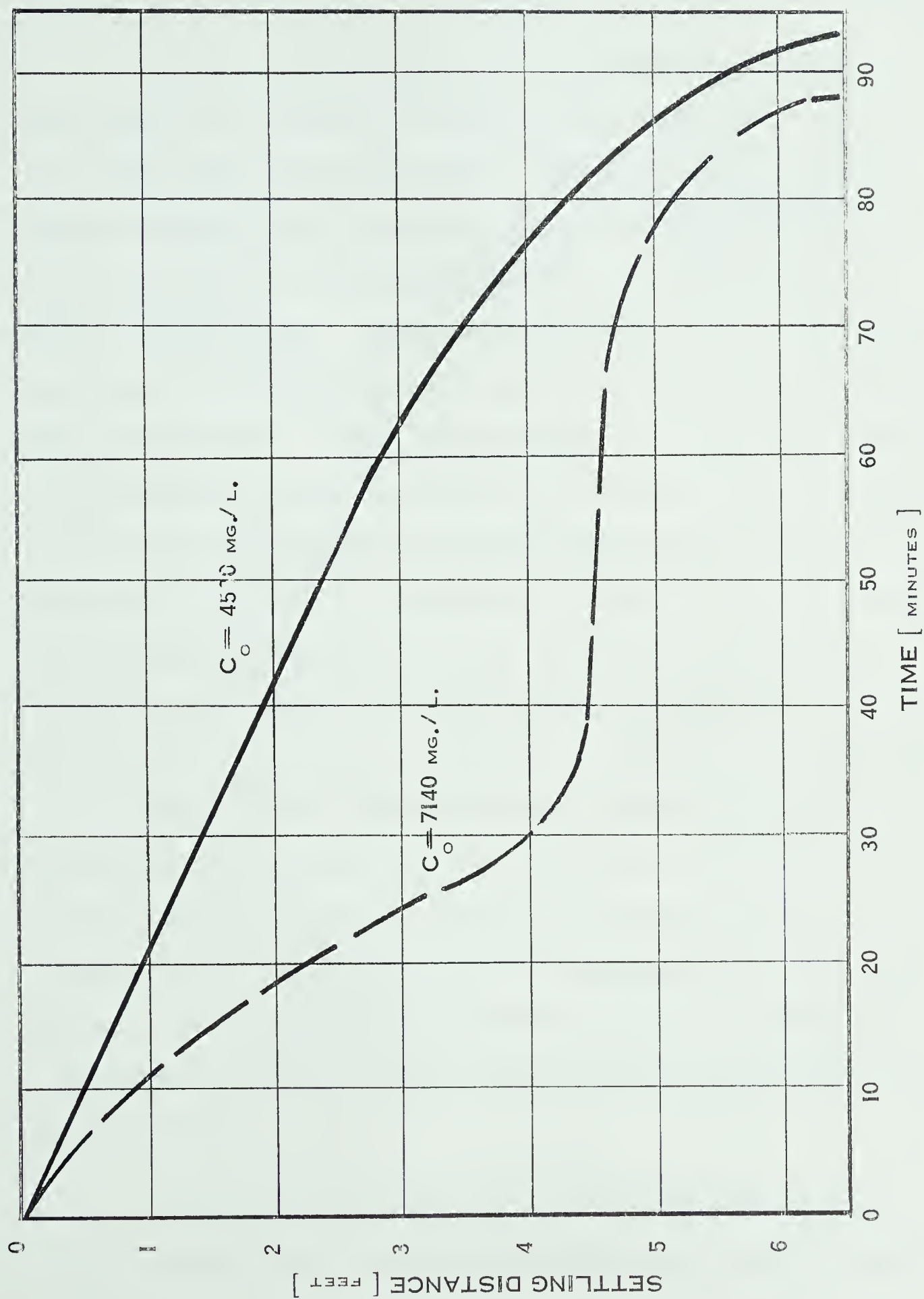


FIGURE : 34 90% SUSPENDED SOLIDS REMOVAL PATHS FOR COMPLETE MIXING ACTIVATED SLUDGE.



are shown with detailed plots of FIGURE 34 in FIGURES 43,44,45, and 46.

From the data of FIGURE 34 it is possible to calculate and show graphically the percent suspended solids removed as a function of detention time and overflow rate. FIGURE 35 indicates a higher suspended solids removal efficiency for the 7140 mg/l MLSS activated sludge than for the 4530 mg/l concentration after approximately 70 minutes detention time. Maximum suspended solids removal efficiencies were found to be 98 percent and 94 percent for the higher and lower MLSS concentrations. FIGURE 36 illustrates that for a MLSS concentration of 7140 mg/l the removal efficiency will be greater than for 4530 mg/l provided the overflow rate is less than 800 Imperial gallons per day. From a practical point of view neither the laboratory derived detention time nor overflow rate should be used in design of sedimentation facilities as the efficiency of the sedimentation process in an actual settling tank is reduced by turbulence and short-circuiting and increased by flocculation due to velocity gradients. Various researchers have discussed these influences in detail and concur that the net effect of these factors results in a decrease of the overflow rate and an increase in the detention time over that derived from the laboratory analysis. As a general rule the overflow rate will be decreased by a factor of 1.25 - 1.75 and the detention period increased by a factor of 1.50 - 2.00.

Using a scale-up factor of 1.75 for detention time and 1.5 for overflow rate, the required detention time for 98% suspended



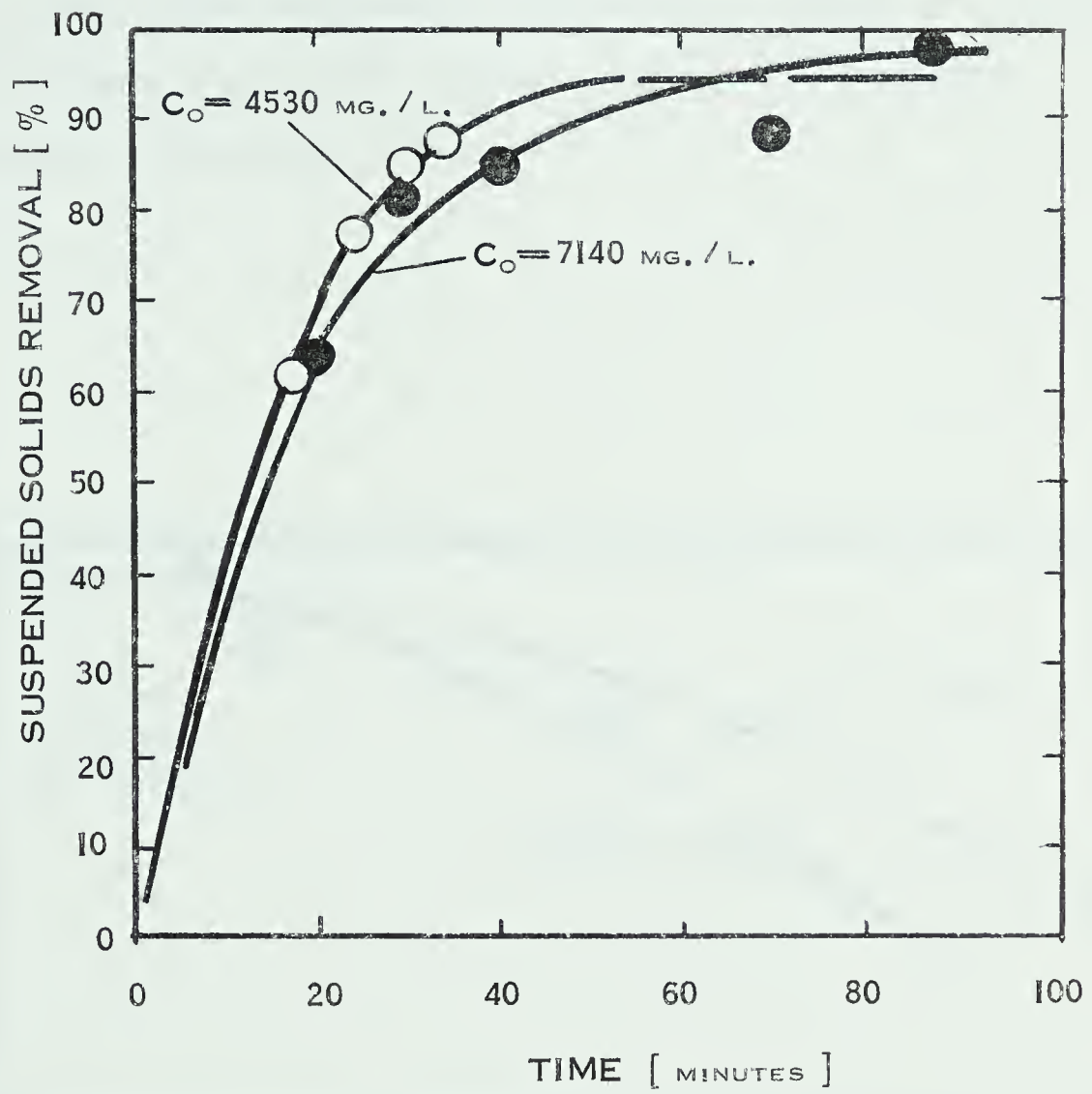


FIGURE : 35 SUSPENDED SOLIDS REMOVAL vs. DETENTION TIME





solids removal from a MLSS concentration approaching 8000 mg/l would be 166 minutes and the overflow rate would be 167 Imperial gallons/day/ft<sup>2</sup>.

Actual data from Phase II is plotted in FIGURE 36 showing 96% MLSS removal at an overflow rate of 156 igpd/ft<sup>2</sup> and MLSS concentration of 2260 mg/l.

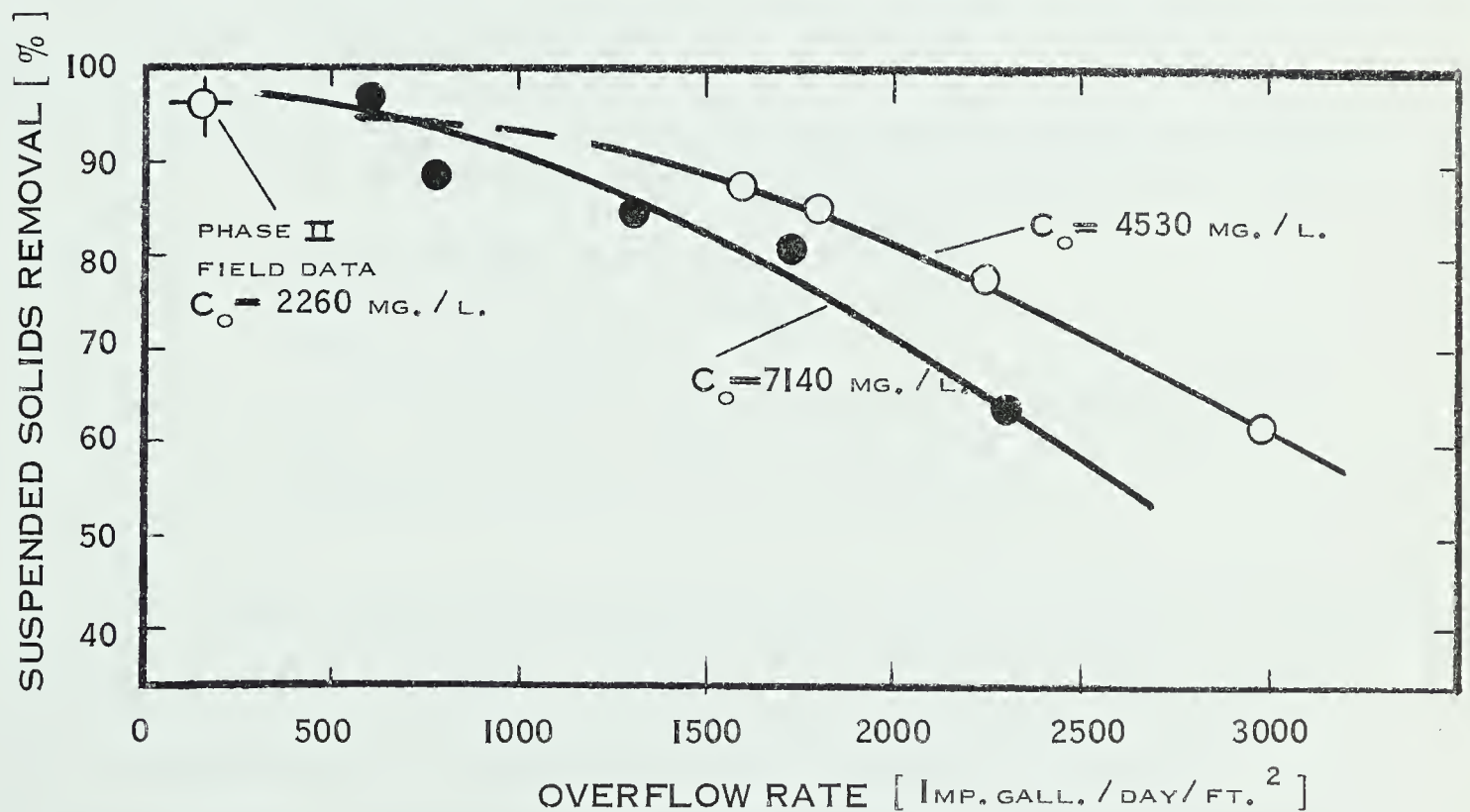


FIGURE : 36 SUSPENDED SOLIDS REMOVAL vs. OVERFLOW RATE



### 3. Oxidation-Reduction Potential Study

Since all chemical reactions involve an exchange of electrons they are considered to be oxidation-reduction reactions. In a reduction-oxidation system electrons flow from the substance oxidized to the substance which is reduced and the potential difference is determined by the ratio of the concentration of oxidant to reductant and the hydrogen ion concentration in accordance with the Nerst equation:

$$E_h = E_o + \frac{RT}{nF} \ln \frac{\text{Oxidant}}{\text{Reductant}}$$

where:

$E_h$  = measured potential in volts referred to standard hydrogen electrode

$E_o$  = a standard potential of the system when the activities of all oxidants and reductants are unity and at 30°C.

$R$  = Gas Constant (8.315 volt coulombs)

$T$  = absolute Temperature. °K

$n$  = no. of electrons that participate in the ORP system

$F$  = Faradays' Constant (95,500 coulombs)

Since activated sludge processes involve biological oxidations of the organic and inorganic components of sewage the same principle is applicable. In an activated sludge, organisms through their enzymes oxidize the substrate BOD giving reduced products and a changing ORP. The rate of change of ORP is an expression of the biological activity or rate of metabolic process. It is this rate of biological activity which is a prime factor in determining the degree of treatment that can be obtained. ORP measurements can not



be used to evaluate an activated sludge process quantitatively but rather serve as a qualitative description of the state of oxidation within the system. This is expressed by the prevailing electrical pressure as a result of the respective concentrations of oxidant and reductant present in the substrate. In measuring the emf in an ORP system an unattackable electrode (platinum, rhodium or gold), constant voltage reference electrode (calomel, silver-silver chloride or hydrogen gas) and a high impedance potentiometer is generally employed. If the hydrogen gas electrode is used then, by convention the potential developed at the measuring electrode can be considered the true potential of the oxidation-reduction reaction being measured. For other electrodes a correction factor must be applied:

$$E_h = E + \text{voltage of reference electrode}$$

where:

E = observed voltage with calomel or silver-silver chloride electrode  
(For calomel electrode  $E = +244\text{mv}$ )

Both pH and temperature affect ORP measurements. Grune (1957) in his study of pH effects on ORP measurements developed an equation taking into account the effect of pH. Eckenfelder and Hood (1951) found that while pH affects ORP by a decrease of 0.6 volts in potential per unit increase in pH, no general effect of pH can be calculated for wastes because of salt effects and even pH affecting the oxidant to reductant ratio. As a general rule ORP becomes more negative with increasing pH.



The effect of temperature on ORP is more pronounced and can be determined as shown in TABLE X prepared by Grune (1957) for raw and digested sludge.

TABLE X

## TEMPERATURE EFFECT ON ORP

Type of Sludge	ORP mv	Sludge Temp. °C
Raw	-175	26
	-155	22
	-90	10
Digested	-200	26
	-165	22
	-125	10

The effect of temperature on the reactivity of raw waste is indicated by the ORP gradient. At colder temperatures the potential remains relatively stable indicating biological inactivity while higher temperatures will increase the potential gradient.

From these observations it is evident that both temperature and pH should be recorded simultaneously with ORP. Nussberger (1953) covers in great detail the practical application of an ORP survey and its implication. Comparative activity curves for activated sludge are presented showing normal and abnormal conditions. The





curves as shown in FIGURE 37 are representative of many such determinations made in connection with laboratory experimentation and plant operation.

Because of a better understanding of ORP and its application, ORP measurements represent an important addition to the tests for successful operation and control of waste treatment plant facilities. In order to determine the biological activity of the Town of Olds oxidation ditch an ORP field survey was conducted. The analysis was performed using equipment as shown in FIGURE 38. A mixed liquor sample was taken in a jar from the ditch. No reaeration of the sample was allowed to take place since the sample was of sufficient volume to permit overflowing after inserting a rubber stopper which contained electrodes etc. as shown. Temperature and pH were recorded at the beginning of the test. The potentiometer circuit was then switched to commence the continuous measurement of the ORP prevailing. In order to maintain homogeneity, the sample was continuously agitated with a magnetic stirrer. The test was conducted at a MLSS concentration of 4530 mg/l, <sup>+</sup>6 degree Centigrade temperature, pH of 7.6 and DO of 0.3 mg/l. For comparative purposes the data obtained was plotted on FIGURE 37.

The initially steep ORP curve obtained is indicative of the high degree of biological activity present even though the liquid temperature was only <sup>+</sup>6 degrees Centigrade. The curve obtained has the appearance of an overloaded condition which would indicate a



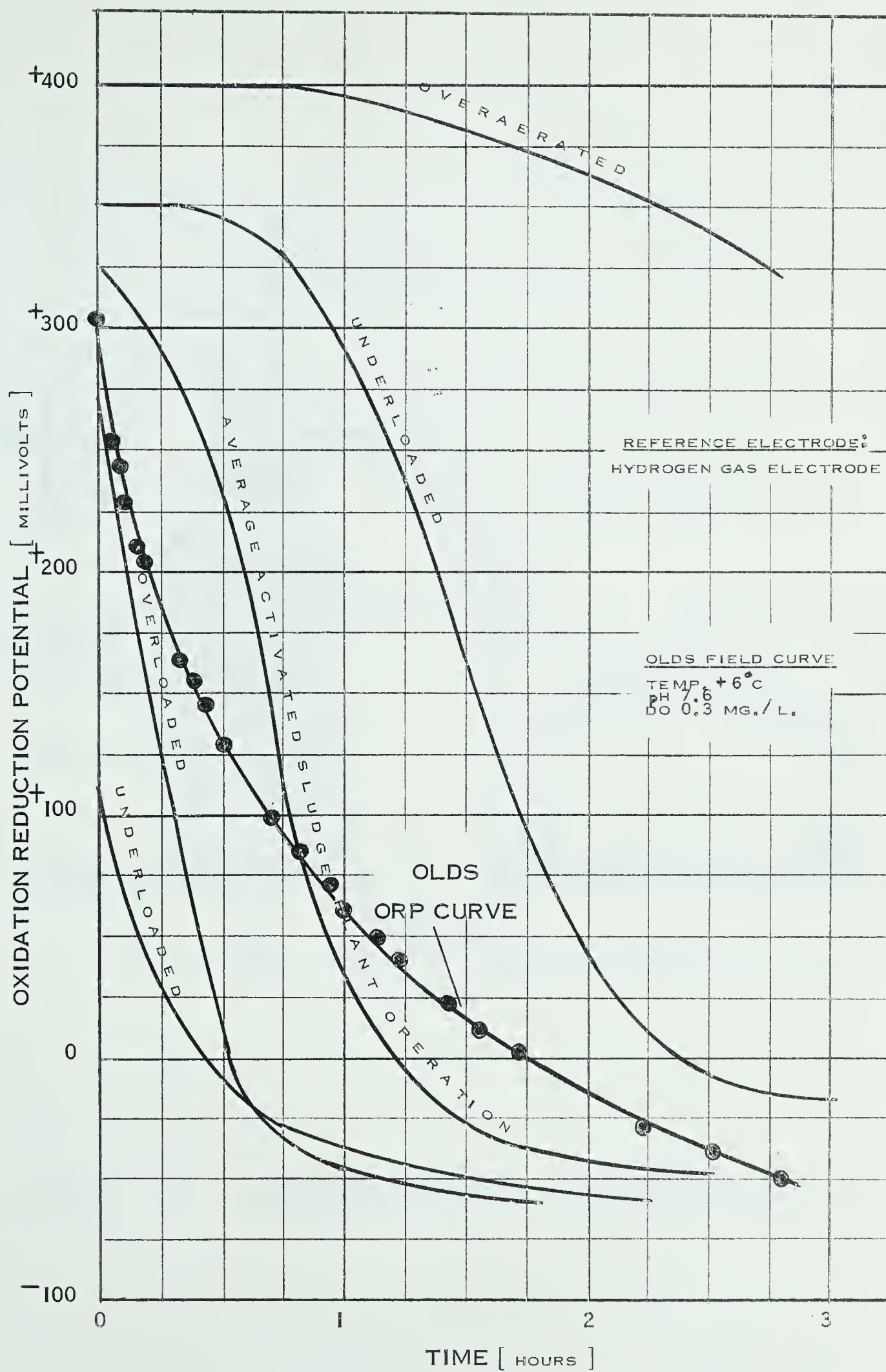


FIGURE : 37 OXIDATION REDUCTION POTENTIAL CURVES



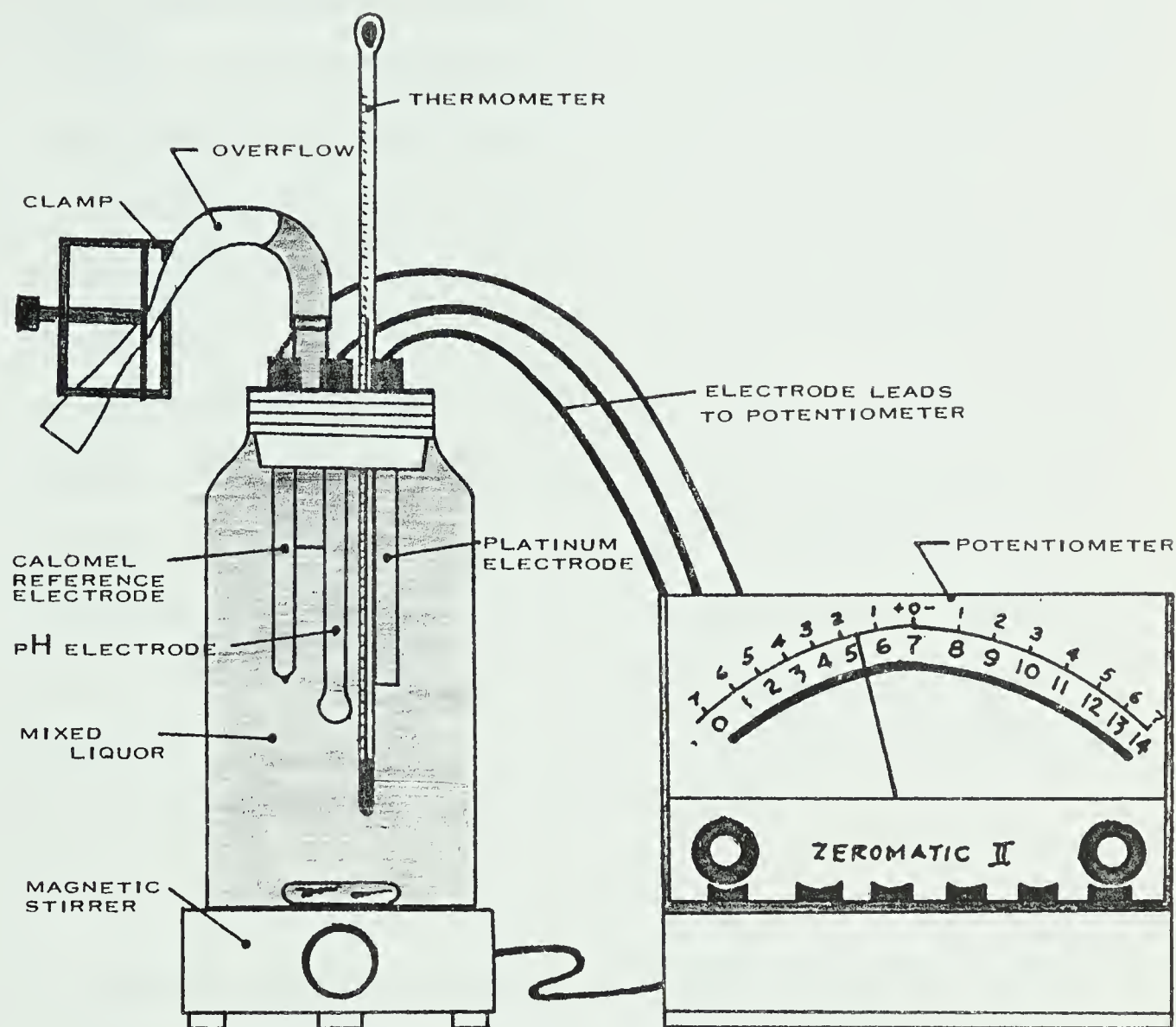


FIGURE : 38      OXIDATION REDUCTION POTENTIAL APPARATUS





high concentration of unoxidized sewage solids in the presence of activated sludge solids. As noted previously the DO content of the mixed liquor sample even though two rotors were operating was only 0.3 mg/l. It can be noted that the biological activity decreases logarithmically and indicates the presence of a stabilizing factor as the curve approaches average activated plant operating conditions. FIGURE 39 shows ORP determinations for a mixed liquor and effluent sample taken February 8th and 6th respectively. The effluent sample when compared to the mixed liquor sample shows a markedly reduced degree of biological activity which is to be expected since the majority of the activated mass has been separated from the mixed liquor and returned to the ditch. The effluent was determined to have a BOD-5 of 44 mg/l with a TSS concentration of 46 mg/l, 89 percent of which were VSS. It would appear that the effluent ORP curve shown is indicative of good treatment.

It was not the objective of this thesis to obtain ORP curves for various operating conditions of oxidation ditches. However, the few curves obtained demonstrate that it would be desirable to conduct extensive studies in order to establish ORP patterns for various operating conditions as noted in FIGURE 37 and use ORP measurements to control the oxidation ditch operation.

#### 4. Oxidation Ditch Mean Velocities

An adequate ditch liquor velocity to maintain the MLSS in suspension is based on the propulsion capacity of the rotor and



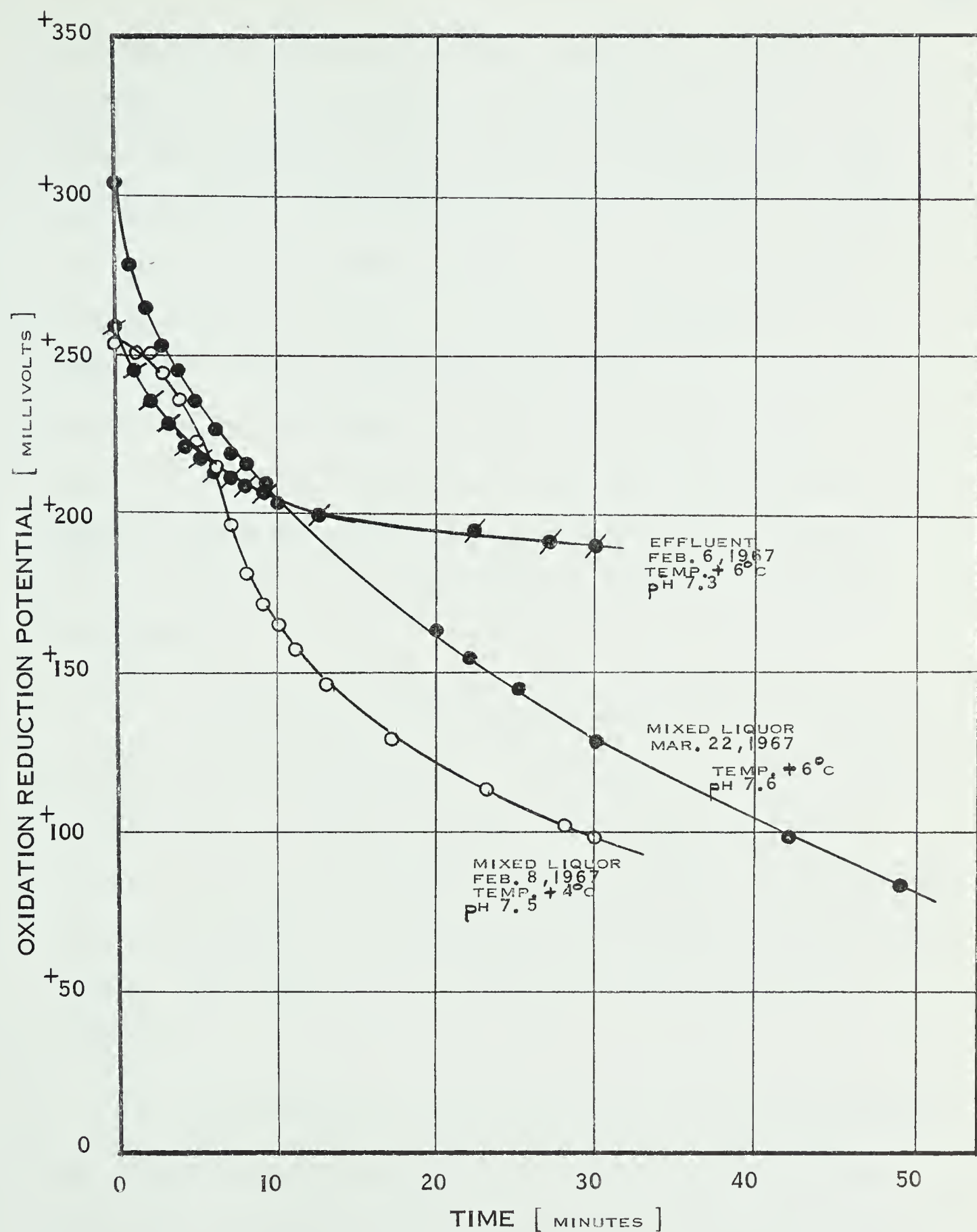


FIGURE : 39 OXIDATION REDUCTION POTENTIAL FIELD CURVES



by the wetted perimeter of the ditch. As noted in CHAPTER II experience had indicated that velocities ranging from 0.8 to 2.0 ft/sec. were required to keep the MLSS from settling out. Based on experience the rotor manufacturer suggests a minimum velocity of 1 ft./sec; to achieve this velocity, a volumetric loading for the rotor of 10,800 l.gal./ft. of rotor has been recommended. From observations made during this study it appears that a 1 ft./sec. velocity may not be adequate. (see FIGURE 17) Since the frictional resistance increases with increased MLSS concentration, thereby lowering the liquid velocity it would be of interest to conduct experiments to determine MLSS concentration vs velocity relationships.

In this study a Gurley-Price current meter was used to determine mean ditch velocities. At the time of the study the MLSS was 4530 mg/l. It was found that a current meter was not very suitable in velocity determinations since the meter bearing assembly had to be dismantled after each 5,30-second runs and stringy matter removed. The results of the velocity survey are summarized in FIGURE 40.

As indicated, the minimum mean ditch velocity with single rotor operation was determined as 1.07 ft./sec. which should be sufficient to maintain the MLSS in suspension. With both rotors operating the mean ditch velocity increased to a minimum of 1.97 ft./sec. at a critical location as shown. Calculations based on



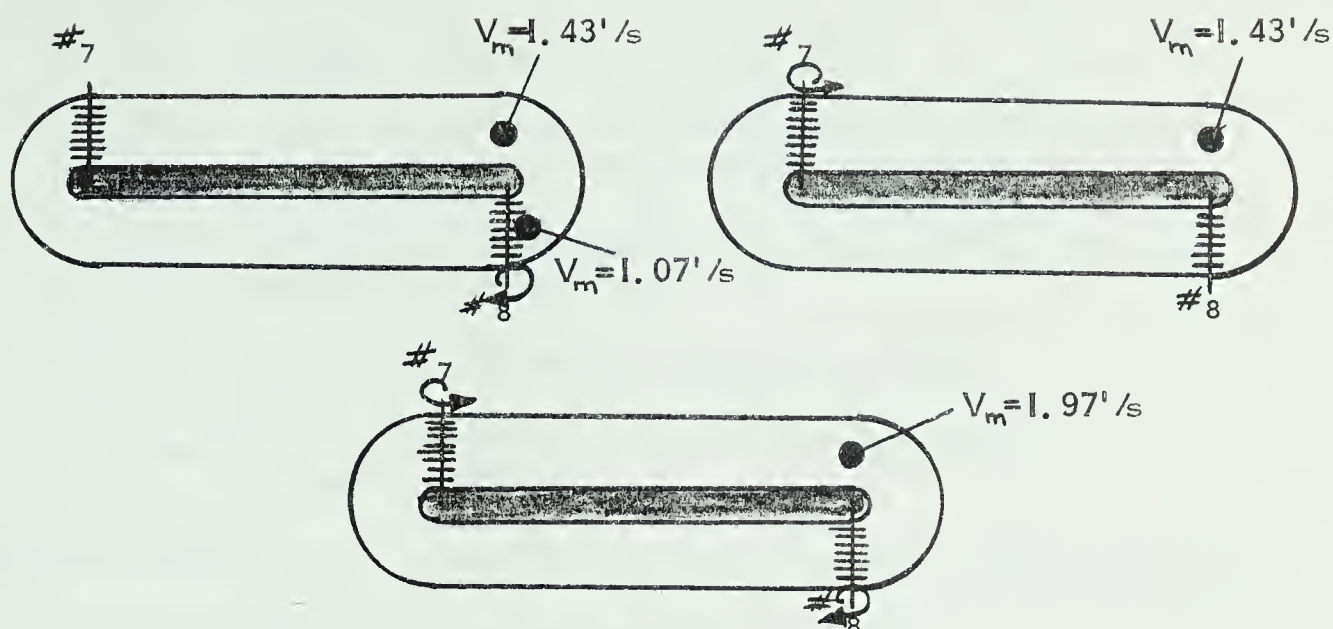


FIGURE : 40 OXIDATION DITCH VELOCITY SURVEY

23 minutes per hour single rotor operation and 37 minutes per hour double rotor operation indicate an average daily mean velocity of 1.54 ft./sec., at a volumetric loading of 12,800 l. gal/ft. of rotor.

#### 5. Oxygen Transfer Rate Coefficient and Oxygen Utilization

The oxygen transfer rate coefficient ( $K_L a$ ) is defined as the oxygen up-take rate divided by the oxygen deficit and refers to the rate that the cage rotors can transfer oxygen to the waste.  $K_L a$  is not related to the rate of biological activity and can be calculated from the following equation:

$$K_L a = \frac{r}{\beta C_s - C_L} = \frac{r}{C_{sw} - C_L}$$

where:

$K_L a$  = oxygen transfer coefficient, per hour





$r$  = oxygen uptake rate, lbs  $O_2$ /hour

$\beta$  = ratio of saturation concentration of DO in waste to saturation concentration of DO in water

$C_s$  = saturation concentration of DO in water at ditch operating temperature

$C_L$  = operating DO level measured in ditch

$C_{sw} = C_s$  = saturation concentration of DO in sewage in ditch.

Using an average ditch velocity of 1.54 ft/second a cross sectional area of 78.1 square feet, and an average DO increase through the rotors of 0.74 mg/l the average oxygen uptake rate was calculated to be 478 lbs/day or 19.9 lbs/hr. Using equation 16 as outlined in CHAPTER III the oxygen uptake rate can be calculated as follows:

$$\begin{aligned}\frac{dO}{dt} &= k_9F + k_2Ma \\ &= 7.5(0.374) + 0.007(508)\end{aligned}$$

$$\frac{dO}{dt} = 6.36 \text{ mg/l/hr.}$$

For a ditch volume of 256,000 Imperial gallons the oxygen uptake rate of 6.36 mg/l/hr corresponds to 16.3 lbs  $O_2$ /hr. This is approximately 18% less than the actual uptake rate of 19.9 lbs  $O_2$ /hr.

The value of  $\beta$  is generally assumed to be 0.9. With a DO saturation concentration of 13.84 mg/l at 36 degrees Fahrenheit for water ( $C_s$ ), the saturation concentration of DO in sewage ( $C_{sw}$ ) in the ditch was calculated to be 12.46 mg/l. The average operating DO level measured in the ditch ( $C_L$ ) was taken from FIGURE 26 as 3.45 mg/l and the oxygen deficit was converted to pounds of oxygen deficit



by multiplying  $(C_{sw} - C_L)$  by 2.56 which was the weight of waste in the ditch in million pounds.  $K_L a$  was then calculated as follows:

$$K_L a = \frac{r}{C_s - C_L} = \frac{19.9}{(12.46 - 3.45)2.56}$$

$$K_L a = 0.86 \text{ per hour}$$

The average  $K_L a$  as calculated compares to average winter and summer  $K_L a$  values of 0.72 and 0.85 per hour determined in a study of the oxidation ditch at Glenwood, Minnesota (Minnesota Department of Health, 1965).

As noted in CHAPTER II aeration equipment is usually rated by manufacturers on the basis of oxygenation capacity (OC). The manufacturers' rating of the Olds cage rotors is 2.75 lbs.  $O_2$  per hour per foot of rotor at operating conditions of 75 rpm and 9 inches immersion. Based on twenty feet of installed rotor, the quoted plant rating is therefore 1320 lbs of  $O_2$  per day. In order to compare the actual number of pounds of oxygen transferred per day to the manufacturers' rotor rating the following equation is used to convert the actual rating to standard conditions:

$$N_o = \frac{K_L a}{\alpha} \times (C_s - 0) \times W \times 24$$

where:

$N_o$  = pounds of oxygen transferred per day at standard conditions

$K_L a$  = oxygen transfer coefficient at 20 degrees Centigrade

$C_s - 0$  = oxygen deficit assuming 0 mg/l initial DO and a  $C_s$  of 9.02 mg/l saturation DO at 20 degrees Centigrade

$\alpha$  = ratio of oxygen transfer in the sewage to oxygen transfer in water

$W$  = weight of sewage under aeration in  $10^6$  lbs.

24 = conversion from hour to day



Since  $K_L a$  is temperature dependent it is corrected to 20 degrees Centigrade by the following relationship:

$$K_L a(T_1) = K_L a(T_2) \theta_R^{T_1 - T_2}$$

The temperature coefficient  $\theta_R$  for the cage rotors has been determined and reported by researchers between 1.016 to 1.047 under controlled laboratory conditions. It appears that the presence of surface active agents and other substances may also affect the temperature coefficient. In this study  $\theta_R$  was assumed to be 1.009 as determined at Glenwood and  $K_L a$  for 20 degrees Centigrade was calculated to be 1.01 per hour. The ratio of oxygen transfer in the sewage to oxygen transfer in water,  $\alpha$  is commonly assumed to be 0.8. At 20 degrees Centigrade the value of  $C_s = 9.02$  mg/l. The weight of sewage under aeration  $W$ , at the time of the study was  $2.56 \times 10^6$  lbs. (at 9 inches immersion). Solving for  $N$ :

$$N_O = \frac{1.01(9.02) 2.56(24)}{0.8}$$

$$N_O = 700 \text{ lbs } O_2 \text{ per day}$$

Based on 20 feet of installed rotor length the actual pounds of oxygen transferred is 1.46 lbs per hour per foot of rotor, as compared to the manufacturers' rating of 2.75 lbs of oxygen per hour per foot of rotor. The Glenwood study also showed an actual oxygen transfer of 1.45 lbs. per hour per foot of rotor vs. 2.75 lbs. quoted. The difference between the actual and quoted oxygen transfer capabilities may in part be attributed to surface aeration which has not been taken into account as well as an increased  $\theta_R$  value and lower  $\alpha$  value. Actual and theoretical oxygen up-take rates and oxygenation capacities





are summarized in TABLE XI

TABLE XI

## OXYGEN TRANSFER RATE COEFFICIENT AND UTILIZATION

Test Program	Oxygen Transfer Coefficient	Oxygen Uptake Rate		Oxygenation Capacity	
		lbs. O <sub>2</sub> /hr.		lbs.O <sub>2</sub> /hr/ft rotor	
	K <sub>L</sub> a/hr.	Theoretical	Actual	Quoted	Actual
Phase II	0.86	16.3	19.9	2.75	1.46

On the basis of the formulae used the actual and theoretical oxygen uptake rates of 19.9 and 16.3 lbs. oxygen per hour are in fair agreement. The rated and actual oxygenation capacity of 1.46 and 2.75 lbs. oxygen per hour per foot of rotor differ considerably and may in part be attributed to inaccuracy of constants used in the various formulae. Based on Phase II testing data the oxygen uptake was computed to be 1.43 lbs. oxygen used per lb. BOD-5 removed. In the design of waste treatment plants a minimum allowance of 1.5 lbs. oxygen per lb. BOD-5 removed is made. The two values are in agreement.



## CHAPTER VII

### SUMMARY AND CONCLUSIONS

This thesis deals with the winter operation of a complete mixing extended aeration waste treatment plant known as an oxidation ditch. Because of the open air exposure of the complete treatment system during ambient temperatures as low as -22 degrees Fahrenheit and limited available knowledge of cold weather operation characteristics of such a system, the performance of the plant was monitored from early November to late February. The testing program was divided into three separate phases: an intermittent testing program extending from October 10th to November 29th (Phase I), a 4-day continuous testing program from December 28th to December 31st (Phase II) and an extended testing program for the period of January 3rd to February 20th (Phase III). During these periods analyses for BOD-5, COD, VSS, MLSS,  $\text{PO}_4^{=}$  and nitrogenous compounds were performed. Special Studies were carried out to determine effluent coliform concentrations, mixed liquor settleability at solids concentrations of 4530 mg/l and 7140 mg/l, oxidation reduction potential of the mixed liquor and treatment plant effluent, ditch liquor velocities, mixed liquor dissolved oxygen concentrations, oxygen transfer rate coefficient and oxygen utilization by the waste.

A mathematical review of complete mixing activated sludge theory was presented and a comparison between calculated and actual



treatment plant performance was made.

After a 2-week plant shut-down during which all sewage bypassed the oxidation ditch, it took 1 week of plant operation before the BOD-5 removal efficiency of the oxidation became 75 percent. FIGURE 27 shows that even at ambient temperatures as low as -22 degrees Fahrenheit the effluent BOD-5 continued to improve, averaging 46 mg/l for the duration of Phase III of the testing program. This indicates that prolonged low temperature operation of the ditch is not detrimental to the biological activity of the treatment system.

Effluent COD values for the complete testing program averaged 218 mg/l. This represents a 60 percent reduction in COD from an average influent COD value of 542 mg/l. Effluent COD values for an oxidation ditch installation at Wierden, Holland were reported by DeJong (1964) as approximately 60 mg/l, representing a COD removal efficiency of that installation of 60 percent. The great influent and effluent COD values indicate a large biologically inert mass. The influent COD/BOD ratio was 1.8 and the effluent COD/BOD ratio 4.75. The difference in COD/BOD ratios is indicative of a high degree of treatment.

As the MLSS concentration in the ditch increased from 2260 mg/l to 3400 mg/l the residual TSS in the effluent decreased from 96 mg/l to 70 mg/l. The increased TSS removal with increased MLSS concentration is substantiated by mixed liquor settleability



studies performed in a settling column. FIGURES 34, 35 and 36 show the result of settling studies of mixed liquor suspensions of 4530 mg/l and 7140 mg/l. Flocculent settling is indicated for a mixed liquor concentration of 4150 mg/l and zone settling preceded by flocculent settling for the 7140 mg/l MLSS concentration. FIGURES 35 and 36 indicate increased suspended solids removal with increasing MLSS concentrations at lower longer detention periods and reduced overflow rates. Actual field data shows that for a MLSS concentration of 2260 mg/l, an overflow rate of  $156 \text{ l.gpd/ft}^2$  and an 8.5 hour detention period, 96% of the MLSS were removed.

FIGURE 23 shows the influent and effluent phosphate concentrations which averaged 5.8 mg/l and 4.6 mg/l respectively. The orthophosphate method was used in the analysis. Since the total phosphate load discharged to a body of water is the sum of the polyphosphates and orthophosphates present in a sewage effluent it is necessary to convert the polyphosphates to orthophosphates before using the orthophosphate method of analysis. FIGURE 30 shows the natural conversion process from polyphosphates to orthophosphates with respect to sample storage time.

As shown by a high  $\text{NH}_3$  effluent concentration, the degree of nitrification at mixed liquor temperatures of  $+2$  degrees Centigrade was negligible throughout the test program. This concurs with the DeJong's (1964) findings at the Wierden oxidation ditch in Holland which operated at  $+5$  degrees Centigrade and also showed





negligible nitrification (DeJong, 1964).

The effluent coliform concentration for samples taken during Phase III of the test program averaged 2000 organisms per ml. This is similar to the effluent coliform concentrations normally found in waste stabilization ponds. Unless the effluent receiving body of water represents a domestic water source the need for effluent chlorination is not indicated.

ORP studies indicated a high degree of biological activity for the mixed liquor at + 6 degrees Centigrade. Effluent ORP values were considerably lower and also indicated a low biological activity. Once field curves for various operating conditions have been established ORP determinations would be valuable in monitoring the operation of an oxidation ditch.

Ditch velocities as shown in FIGURE 40 were found to vary from 1.07 ft/sec for single rotor operation to 1.97 ft/sec for double rotor operation. Both rotors are on time-clocks and operate together for 37 minutes per hour and individually for 23 minutes per hour resulting in an average daily mean velocity of 1.54 ft/sec. At a ditch volume of 256,000 Imperial gallons and 20 feet of rotor, the hydraulic load on the rotor is 12,800 I. gal./ft. of rotor. This compares to the rotor manufacturers' rating of 10,800 I.gal/ft of rotor in order to achieve a 1.0 ft/sec. velocity. The appearance of a sludge bar due to low liquid velocities at the inside of one



of the oxidation ditch bends suggests that the flow pattern around the bends of the oxidation ditch must be controlled.

Field studies indicate that 19.9 lbs oxygen per hour are required by the waste. This compares to a theoretical value of 16.3 lbs oxygen per hour as calculated from McKinney's (1962) mathematical formulations. The field study showed that 1.43 lbs oxygen are required by the waste for each pound of BOD-5 removed. This is in agreement with design allowances of 1.5 lbs oxygen required per lb of BOD-5 removed.

FIGURE 25 shows the dissolved oxygen variations in the ditch which varied throughout the day from a maximum of 6 mg/l at 8:30 a.m. to a low of 1.8 mg/l at 8:30 p.m. The average daily effluent dissolved oxygen was 3.5 mg/l. Under single rotor operation the dissolved oxygen concentration was approximately 1.0 mg/l. Under these conditions the effluent dissolved oxygen concentration was 0.5 mg/l.

The oxygen transfer rate coefficient,  $K_L a$ , was calculated to be 0.86 per hour in comparison to 0.72 at the Glenwood, Minnesota oxidation ditch. While for both Olds and Glenwood the rated oxygenation capacity of the rotors under identical conditions (9" immersion and 75 rpm) is 2.75 lbs oxygen per hour per foot of rotor, the actual uptake rates were determined as 1.46 and 1.45 lbs oxygen per hour per foot of rotor. This difference between the actual and quoted



oxygenation capacities may in part be attributed to natural surface aeration which has not been taken into account as well as a temperature coefficient ( $\theta_R$ ) greater than 1.009 used and a lower oxygen transfer in sewage to oxygen transfer in water ratio ( $\alpha$ ).

As noted in TABLE VII the MLVSS concentration is not a good measure of the active microbial mass in the oxidation ditch. The calculations indicate that even though the MLVSS increase, from 1540 mg/l to 2195 mg/l the percent active mass decreases from 33% to 22%.

Calculated BOD-5 effluent values as shown in TABLE VII indicate that a BOD-5 concentration of less than 20 mg/l should be expected. Actual effluent BOD-5 values are 100 percent higher than theoretical. No reliance should be placed on theoretical calculations for low temperature operation until more information for k-factors at low temperatures becomes available. If, lacking better data, a theoretical approach is used for effluent BOD-5 prediction at low temperatures, the calculated BOD-5 value should be multiplied by a factor of 2. The validity of the theoretical analyses during oxidation ditch summer operation remains to be investigated.

TABLE VII shows the great difference between theoretical and actual effluent VSS. For a theoretical VSS effluent value of 125 mg/l the actual VSS in the effluent were 55 mg/l. TABLE VII





shows that the actual VSS to BOD-5 ratio is between 0.9 and 1.5 and not around 7.0 as theoretically calculated. Further studies over a longer time interval are required to verify the contention that a high suspended solids - low BOD-5 effluent is typical of extended aeration systems (McKinney, 1962).

These studies have shown that the oxidation ditch is a suitable method for waste treatment during severe winter conditions in Western Canada.



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## APPENDIX A

## Supplementary Figures



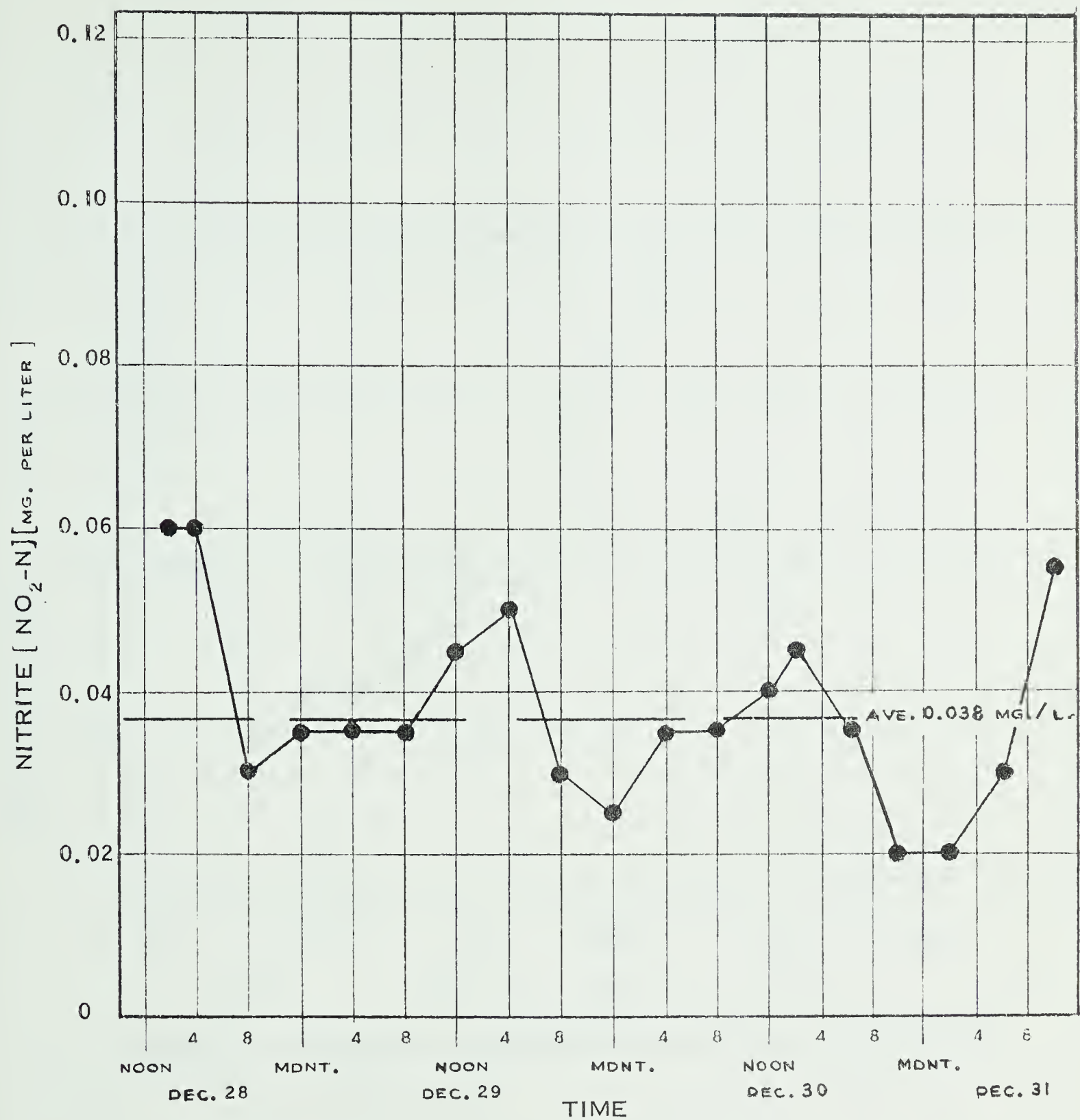


FIGURE : 41 DIURNAL EFFLUENT NITRITE [ NO<sub>2</sub> - N ] VARIATIONS



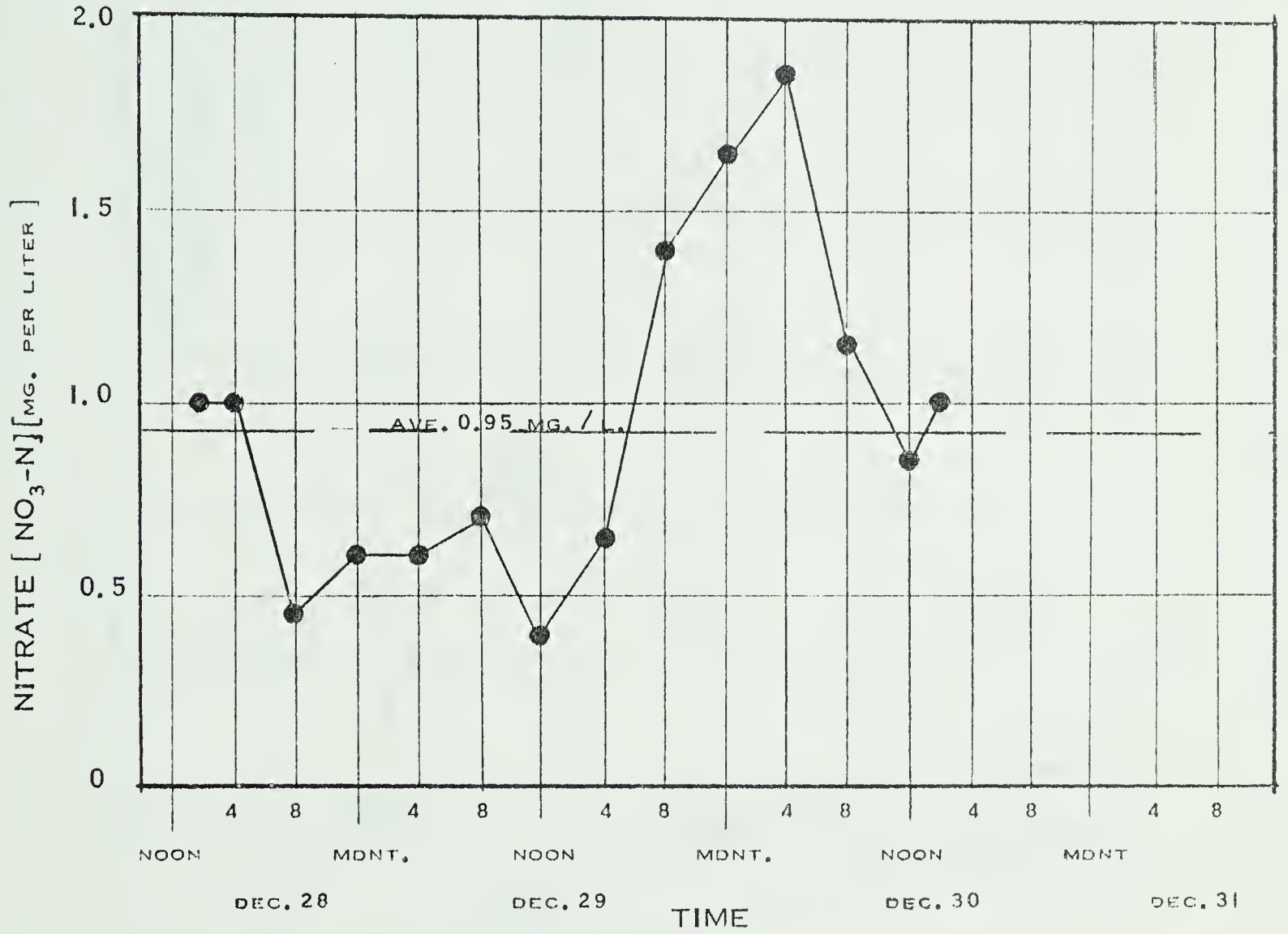


FIGURE : 42 DIURNAL EFFLUENT NITRATE [NO<sub>3</sub>-N] VARIATIONS





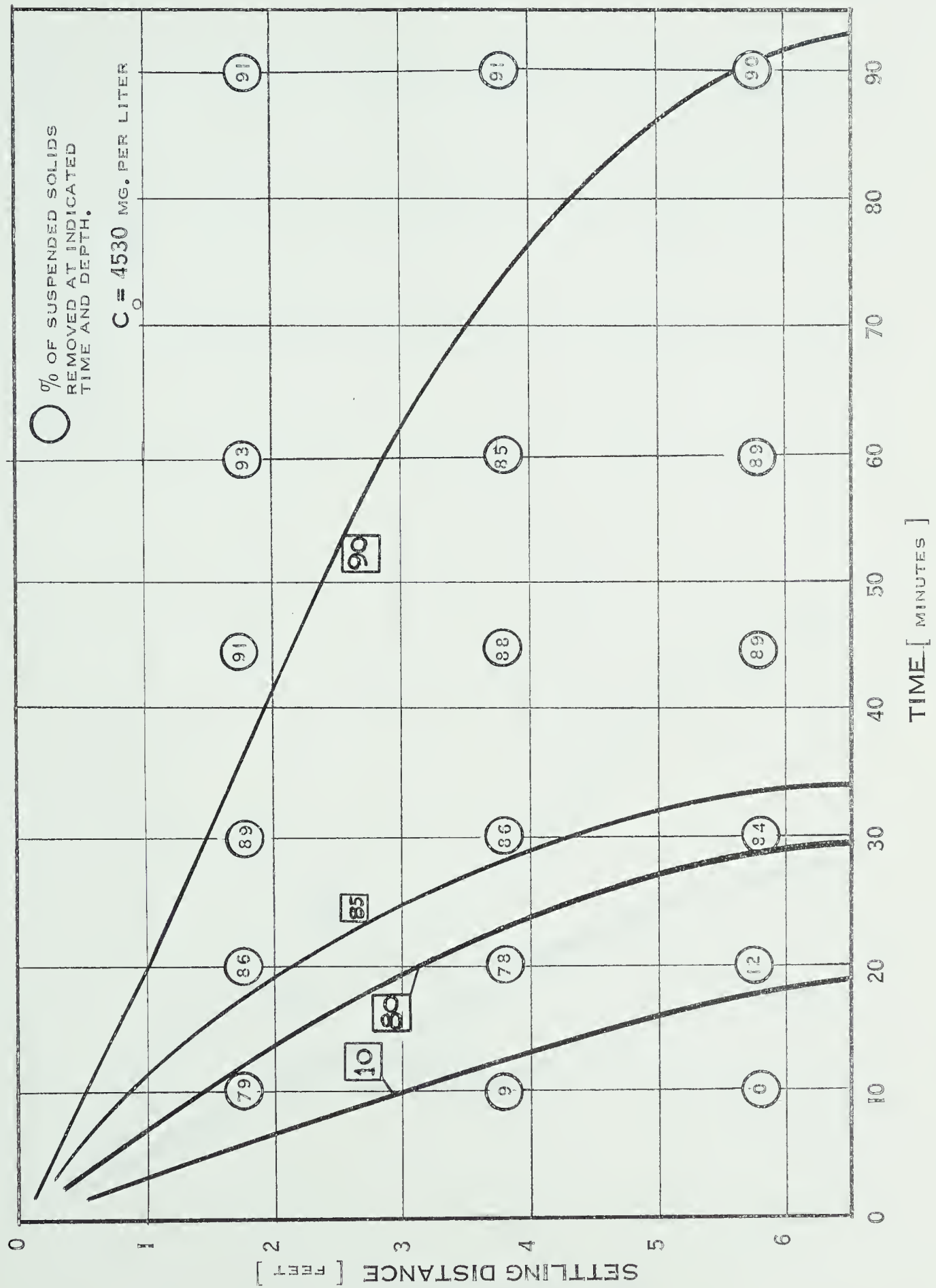
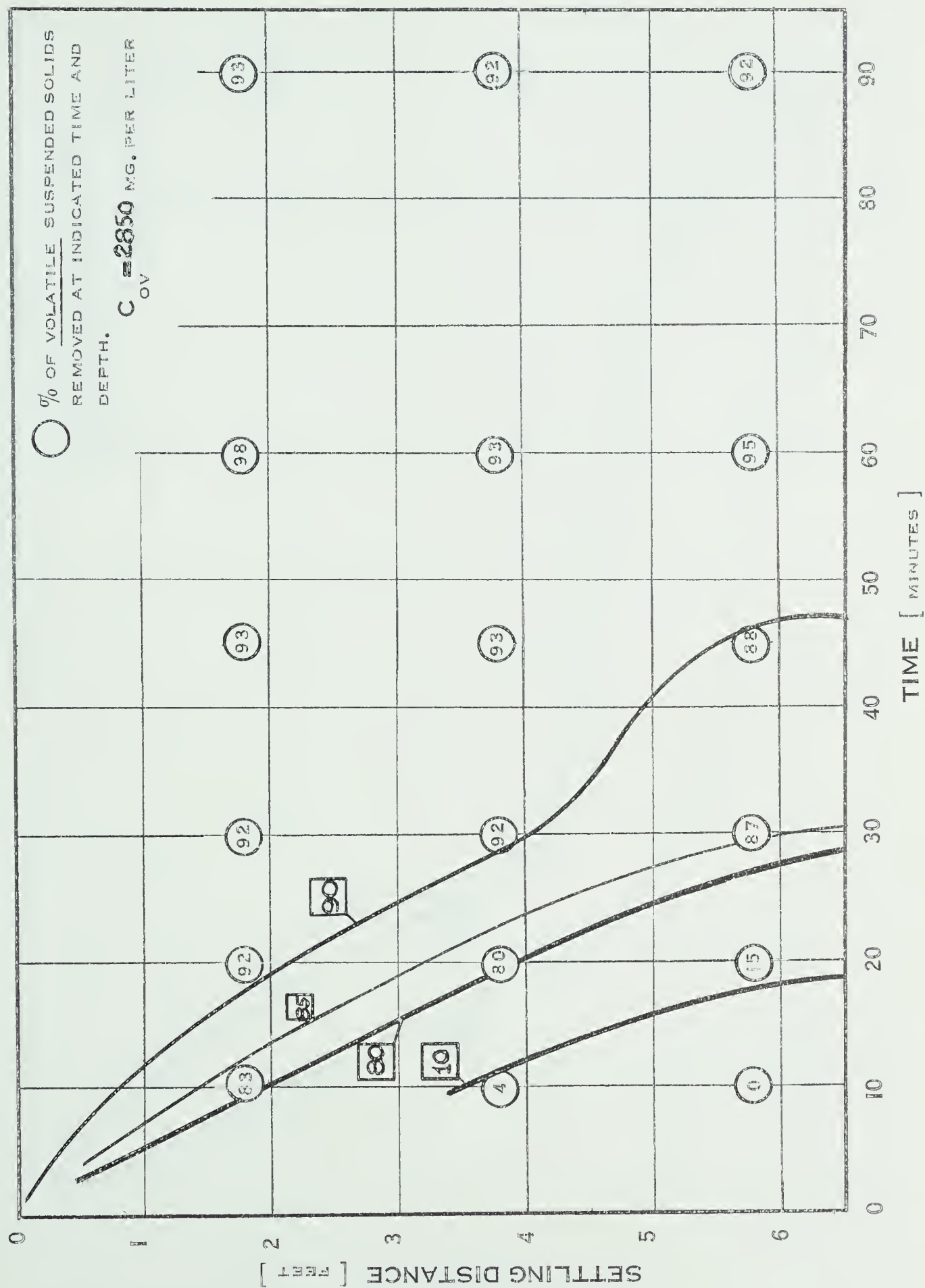


FIGURE: 43 SETTLING PATHS FOR COMPLETE MIXING ACTIVATED SLUDGE







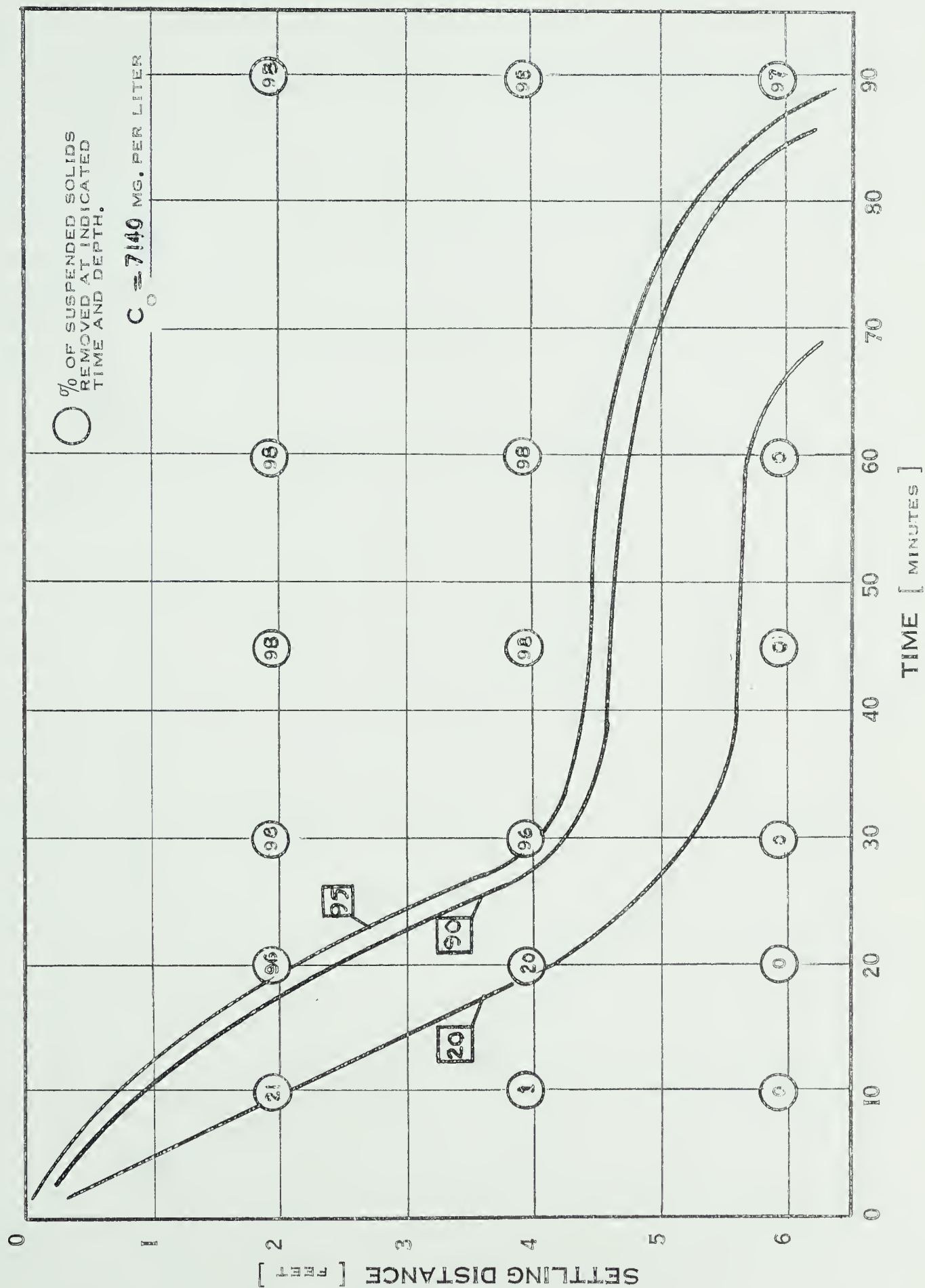


FIGURE : 45 SETTLING PATHS FOR COMPLETE MIXING ACTIVATED SLUDGE





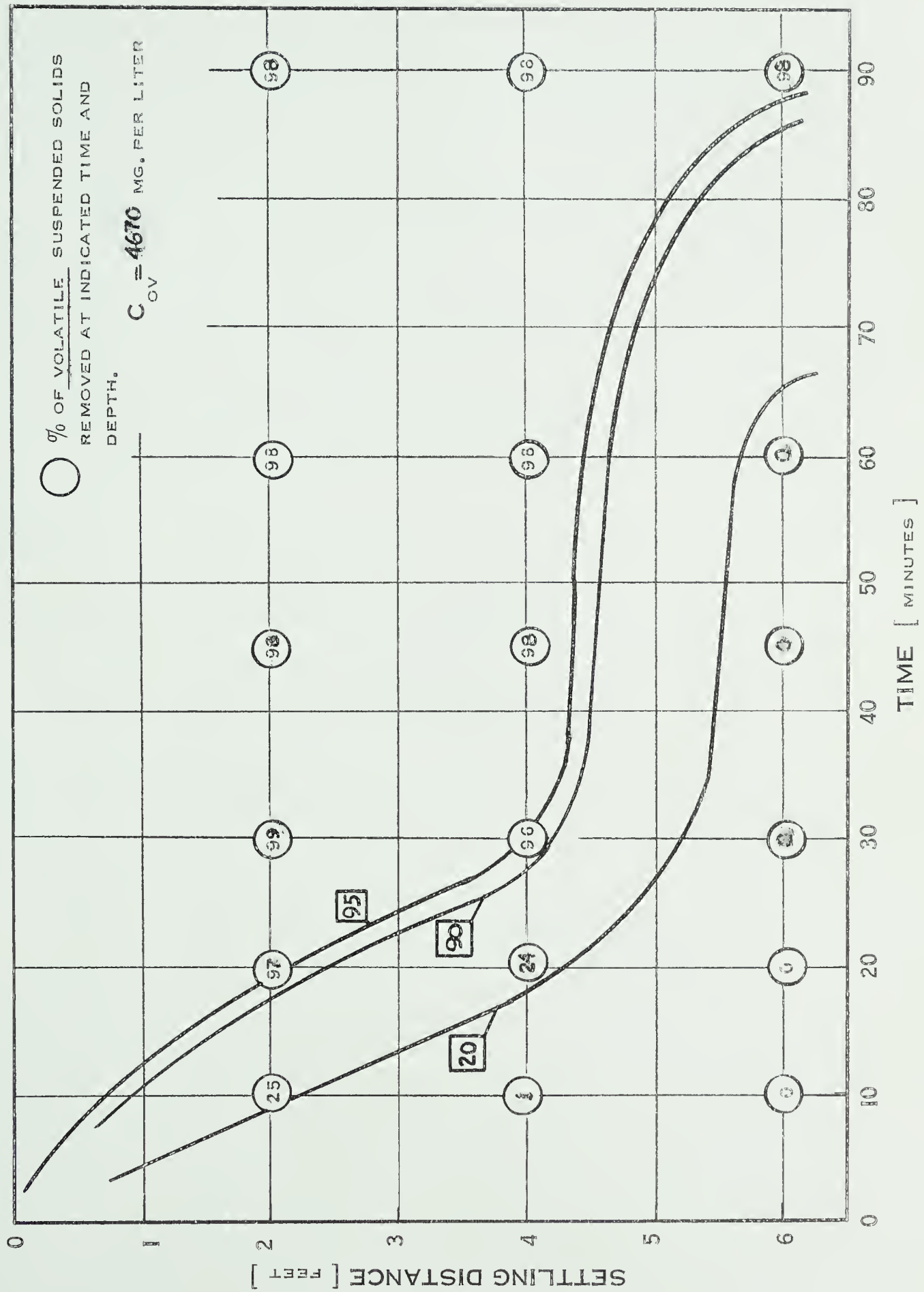


FIGURE: 46 SETTLING PATHS FOR COMPLETE MIXING ACTIVATED SLUDGE  
[ VOLATILE FRACTION ONLY ]





## APPENDIX B

## Supplementary Tables



TABLE XII

k - FACTORS FOR COMPLETE MIXING ACTIVATED SLUDGE  
(McKinney, 1962)

FACTOR	DESCRIPTION	VALUE
$k_1$	Synthesis Constant	0.5
$k_2$	Endogenous Respiration Constant ( at 20 degrees Centigrade)	0.007
$k_5$	Metabolic Rate Constant (mg/l ultimate oxygen demand removed per mg/l ultimate oxygen demand remaining per day)	360
$k_6$	Synthesis Constant (for F in BOD-5)	250
$k_7$	Endogenous Respiration Constant (related to $k_2$ , mg/l decrease in active mass per mg/l active mass per day)	0.144
$k_8$	Endogenous Metabolism Constant (@20 degrees Centigrade, per day)	0.036
$k_9$	Oxygenation Constant (mg/l oxygen utilization per mg/l BOD-5 remaining per hour)	7.5
$k_{10}$	Oxygenation Constant (for carbonaceous metabolism at 20 degrees Centigrade over 5 days)	0.6



TABLE XIII

## TOWN OF OLDS POLLUTION CONTROL CENTRE DESCRIPTION

## Oxidation Ditch:

Length	- 570 lineal feet
Width	- 10 feet (bottom width)
Side Slopes	- 2:1
Depth	- variable 4'-0" to 5'-0"
Median Strip	- 32 feet wide

## Rotors:

Type	- Cage Rotor
Number	- 2
Length	- 10'-0" each
Diameter	- 27½"
Horsepower	- 10
Rotational Speed	- 75 rpm and 98 rpm

## Settling Tanks:

Type	- Mechanized
Length	- 60 feet
Width	- 16 feet
Depth	- 9 feet and 10 feet

## Sludge Drying Bed:

Area	- 1850 square feet
------	--------------------

## Sludge Lagoons:

Number	- 3
--------	-----

## Sludge Storage:

Type	- Abandoned Imhoff Tank
Volume	- 6,600 cubic feet





TABLE XIV

## EFFLUENT DISSOLVED OXYGEN SURVEY

DATE	TIME	D.O.Rdg. mg/l	ELAPSED TIME hrs.
Dec. 28/66	8:30 p.m.	1.8	0.5
Dec. 29/66	4:30 a.m.	1.4	8.5
	6:30 a.m.	3.3	10.5
	7:30 a.m.	2.0	11.5
	NOON	3.7	16.0
	2:00 p.m.	2.2	18
	4:00 p.m.	2.7	20
	6:00 p.m.	1.0	22
	8:00 p.m.	1.3	24
	MDNT	1.3	28
Dec. 30/66	2:00 a.m.	1.7	30
	4:00 a.m.	2.1	32
	8:00 a.m.	3.5	36
	10:00 a.m.	2.8	38
	NOON	3.5	40
	2:00 p.m.	3.3	42
	4:00 p.m.	2.7	44
	6:00 p.m.	2.7	46
	10:00 p.m.	1.9	50
	MDNT	1.9	52
Dec. 31/66	2:00 a.m.	1.5	54
	6:00 a.m.	2.6	58
	8:00 a.m.	3.0	60
	10:00 a.m.	3.0	62



TABLE XV

## MIXED LIQUOR DISSOLVED OXYGEN

Station	Dec. 28/66 8:30 pm	Dec. 29/66 4:30 am	Dec. 30/66 12:30 am	Dec. 30/66 8:30 am	Dec. 30/66 12:30 pm	Dec. 30/66 4:30 pm
0 00	1.4	3.3	2.2	6.0	3.9	1.4
0 55	2.1	3.0	3.0	5.5	4.3	2.0
0 75	2.7	3.8	3.6	5.9	5.2	2.9
2 30	1.6	3.2	2.5	5.8	4.0	2.2
2 85	1.5	3.1	2.5	5.8	3.9	1.9
3 40	1.6	3.1	3.0	5.8	3.9	2.1
3 60	2.5	3.6	3.9	6.3	4.9	3.0
5 15	1.3	3.0	2.7	5.8	4.0	2.3
5 70	1.4	3.3	2.2	6.0	3.9	1.4
Ave	1.8	3.3	2.9	5.9	4.3	2.5

All values in mg/l.



TABLE XVI

## INFLUENT GRAB SAMPLE ANALYSES - Phase II

SAMPLE NO.	DATE	TIME	BOD	COD	TSS	VSS	TS	TVS	PO <sub>4</sub>	NH <sub>3</sub> -N
1	28.12	1:30 pm	355	575	465	350	1869	541	24	32
2		4:00 pm	440	619	415	335	1957	624	17	31.8
6		8:00 pm	380	504	200	177	1580	344	6.2	29.5
7	28.12	MDNT	290	345	158	145	1500	275	3.9	29.0
8	29.12	4:00 am	205	239	112	104	1399	192	1.0	33.0
13		8:00 am	100	96	38	32	1154	110	0.9	20.5
14		NOON	410	691	410	335	1798	485	8.0	30.5
15		4:00 pm	375	662	310	300	1806	462	5.1	26.0
21		8:00 pm	350	653	215	177	1661	378	5.5	39.0
22	29.12	MDNT	285	548	143	135	1538	292	3.9	37.0
23	30.12	4:00 am	160	308	73	65	1336	185	2.2	43.0
28		8:00 am	110	144	38	35	1127	96	0.8	23.0
29		NOON	380	827	340	272	1737	465	4.2	52.0
30		2:00 pm	375	933	370	330	1725	414	3.4	34.0
37		6:00 pm	465	1050	450	390	1887	550	10.0	35.0
38	30.12	10:00 pm	410	913	356	292	1793	515	5.4	39.0
39	31.12	2:00 am	250	612	292	218	1535	344	3.8	38.0
40		6:00 am	135	233	68	48	1242	150	1.8	30.5
41		10:00 am	175	340	124	112	1382	247	3.1	57.5

All values in mg/l.



TABLE XVII

## EFFLUENT GRAB SAMPLE ANALYSES - Phase II

SAMPLE NO.	DATE	TIME	BOD	COD	TSS	VSS	TS	TVS	PO <sub>4</sub>	NH <sub>3</sub> -N
3	28.12	1:30 pm	74	221	160	130	1480	245	8.8	33.0
4		4:00 pm	67	150	100	75	1384	185	9.5	33.0
9		8:00 pm	84	221	90	82	1405	177	3.9	29.5
10	28.12	MDNT	90	221	92	88	1436	187	4.4	29.5
11	29.12	4:00 am	111	259	164	130	1484	225	3.8	28.0
17		8:00 am	77	221	100	73	1238	140	5.1	24.5
18		NOON	75	221	77	67	1369	150	5.0	23.8
19		4:00 pm	78	250	108	78	1312	171	6.0	23.8
24		8:00 pm	80	269	128	105	1397	177	3.8	29.0
25	29.12	MDNT	62	269	85	72	1276	173	3.9	30.5
26	30.12	4:00 am	65	240	80	55	1422	174	3.8	32.0
32		8:00 am	63	192	88	56	1273	148	3.9	27.5
33		NOON	68	240	124	52	1399	145	3.8	27.5
34		2:00 pm	70	221	92	52	1339	98	3.9	29.5
43		6:00 pm	65	205	80	56	1340	148	3.8	31.5
44	30.12	10:00 pm	65	224	68	40	1413	160	3.9	32.0
45	31.12	2:00 am	67	214	64	52	1436	155	3.2	32.0
46		6:00 am	63	195	80	64	1439	145	3.9	30.5
47		10:00 am	69	205	40	24	1419	146	3.5	30.5

All values in mg/l.





TABLE XVIII

## COMPOSITE SAMPLE ANALYSES - Phase II

TEST	MIXED LIQUOR		EFFLUENT	
	Dec. 29, 1966 Composite No. 1 Sample No. 12	Dec. 30, 1966 Composite No. 2 Sample No. 27	Dec. 29, 1966 Composite No. 1 Sample No. 20	Dec. 30, 1966 Composite No. 2 Sample No. 35
BOD	505	610	70	65
COD	2240	2370	231	221
NH <sub>3</sub> -N	23.0	26.5	25.0	29.5
NO <sub>2</sub> -N	0.02	0.02	0.04	0.025
NO <sub>3</sub> -N	0.6	1.85	0.55	0.85
Total N	133.3	132.2	55.4	49.3
PO <sub>4</sub> <sup>=</sup>	8.0	8.5	5.0	3.8
Cl <sup>-</sup>	125	125	135	145
TSS	2330	2190	123	92
SVS	1580	1500	95	52
SFS	750	690	28	40
TS	3621	3351	1232	1293
VS	1698	1461	168	117
FS	1923	1890	1064	1176

All values in mg/l.



TABLE XIX

## EFFLUENT SAMPLE ANALYSES - Phase III

Sample No.	Date	BOD	COD	TSS	VSS	PO <sub>4</sub>	NH <sub>3</sub> N
-	28/12	-	221	-	94	-	-
-	29/12	67	221	-	73	-	-
-	30/12	65	218	-	56	-	-
49	3/1	39	194	85	82	40	47
50	4	36	203	75	72	44	45
51	5	36	203	65	61	45	46
52	6	45	213	123	118	44	42
53	7	42	184	55	50	44	47
54	8	39	205	73	73	44	48
55	9	38	211	42	42	40	50
56	11	42	221	34	34	48	46
57	12	41	221	64	64	44	43
58	13	41	221	44	44	45	46
59	14	60	230	54	54	44	47
60	16	58	230	48	48	44	49
61	18	73	362	204	120	49	35
62	19	49	257	134	78	49	39
63	20	48	267	126	74	44	38
64	23	39	248	92	60	44	42
65	25/1	41	257	98	64	48	37
66	2/2	39	200	84	50	42	40
67	6	41	182	46	41	44	39
68	7	45	182	48	48	44	42
69	8	49	200	52	52	47	40
73	9	31	138	24	4	42	38
74	10	35	166	34	24	42	41
75	11	39	166	28	12	45	43
76	13	33	230	52	26	39	37
77	14	49	239	76	48	44	42
78	16	44	193	24	12	48	45
79	17	54	239	90	48	44	43
80	18	57	166	80	62	45	39
81	20/2	57	248	42	34	44	39

Note: only shown values plotted.

All values in mg/l.



APPENDIX C

List of Abbreviations





## LIST OF ABBREVIATIONS

BOD-5	Five-day biochemical oxygen demand
$\text{Cl}^-$	Chloride ion
COD	Chemical oxygen demand
DO	Dissolved oxygen
F/M	Food to organism ratio
FS	Fixed solids (total)
FSS	Fixed suspended solids
ML	Mixed liquor
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
$\text{NH}_3\text{-N}$	Ammonia nitrogen
$\text{NO}_2\text{-N}$	Nitrite nitrogen
$\text{NO}_3\text{-N}$	Nitrate nitrogen
OC	Oxygenation capacity
ORP	Oxidation reduction potential
$\text{PO}_4^{=}$	Phosphate ion
SVI	Sludge volume index
Total N	Total nitrogen
TS	Total solids
TSS	Total suspended solids
TVS	Total volatile solids
TVSS	Total volatile suspended solids











**B29865**